

chapter:-3 (precipitation)

flake → अणु

The term precipitation denotes all forms of water that reach the earth from the atmosphere in the form of solid or liquid such as rainfall, dew, frost, hail, snowfall, etc.

3.1) Causes, Occurrence & Forms of precipitation
Some of the common forms of precipitation are:-
rain, snow, drizzle, glaze, sleet & hail.

a) Rain

It is the principal form of precipitation in Nepal. The term rainfall is used to describe precipitations in the form of water drops of sizes larger than 0.5mm. Any drop larger in size than this tends to break up into drops of smaller sizes during its fall from the clouds.

b) Snow

Snow is another important form of precipitation. Snow consists of ice-crystals which usually combine to form flakes. In Nepal, snow occurs only in the Himalayan regions.

c) Drizzle

A fine sprinkle of numerous water droplets of size less than 0.5mm and intensity less than 1mm/h is known as

sleet \Rightarrow असिना

drizzle. In this the drops are so small that they appear to float in the air.

d) Glaze

When rain or drizzle comes in contact with cold ground at around 0°C , the water drops freeze to form an ice coating called glaze or freezing rain.

e) Sleet

It is frozen raindrops of transparent grains which form when rain falls through air at subfreezing temperature.

f) Hail

It is a showery precipitation in the form of irregular pellets or lumps of ice of size more than 8 mm.

3.2) Rainfall Measurements

Precipitation is expressed in terms of the depth to which rainfall water would stand on an area if all the rain were collected on it. Thus 1 cm of rainfall over a catchment area of 1 km^2 represents a volume of water equal to 10^4 m^3 . In the case of snowfall, an equivalent depth of water is used as the depth of precipitation. The precipitation is collected & measured in a raingauge. Terms such as pluviometer, ombrometer & hyetometer are also sometimes used to

designate a raingauge.

A raingauge essentially consists of a cylindrical vessel assembly kept in the open to collect rain. Raingauges can be broadly classified into two categories as

- i) nonrecording raingauges &
- ii) recording raingauges.

i) Nonrecording raingauge.

In india, it is also called symon's gauge. It essentially consists of a circular collecting area of 12.7 cm (5.0 inch) diameter connected to a funnel. The funnel discharges the rainfall catches into a receiving vessel. The funnel & receiving vessel are housed in a metallic container. Water contained in the receiving vessel is measured by a suitably graduated measuring glass, with an accuracy up to 0.1 mm. The receiving bottle normally doesnot hold more than 10 cm of rain & as such in the case of heavy rainfall the measurements must be done more frequently & entered. Proper care, maintenance & inspection of raingauges, espically during dry weather to keep the instrument free from dust & dirt is very necessary.

This raingauge can also be used to measure snow-fall. When snow is expected, the funnel & receiving bottle are removed & the snow is atted & allowed to collect to the outer metal container. The snow is then melted & the depth

actuate \Rightarrow to activate . or to put into motion.

of resulting water is measured. Antifreeze agents are sometimes used to facilitate melting of snow.

ii) Recording raingauge

Recording raingauge produce a continuous plot of rainfall against time & provide valuable data of intensity & duration of rainfall for hydrological analysis of streams. The following are some of the commonly used recording raingauges.

a) Tipping - Bucket type

This consist of a cylindrical receiver 30 cm diameter with a funnel inside. Just below the funnel a pair of tipping buckets is pivoted such that when one of the bucket receives a rainfall of 0.25 mm it tips & brings the other one in position. The water from the tipped bucket is collected in a storage can. The tipping moves the electrically driven pen to trace a record on clock-work driven chart. This type cannot record snow.

b) Weighting Bucket type

In this type of rain-gauge, when a certain weight of rainfall is collected in a tank, which rest on a spring-lever balance, it makes pen to move on a clock-work driven chart. The rotation of drum sets the time scale while

the vertical motion of the pen records the cumulative precipitation.

3.3) Presentation of Rainfall Data

A few commonly used methods of presentation of rainfall data which have been found to be useful in interpretation and analysis of such data are given as follows.

a) Mass Curve of rainfall

The mass curve of rainfall is a plot of the accumulated precipitation against time, plotted in chronological order. Records of weighing bucket type gauges are of this form. A typical mass curve of rainfall at a station during a storm is shown in figure below.

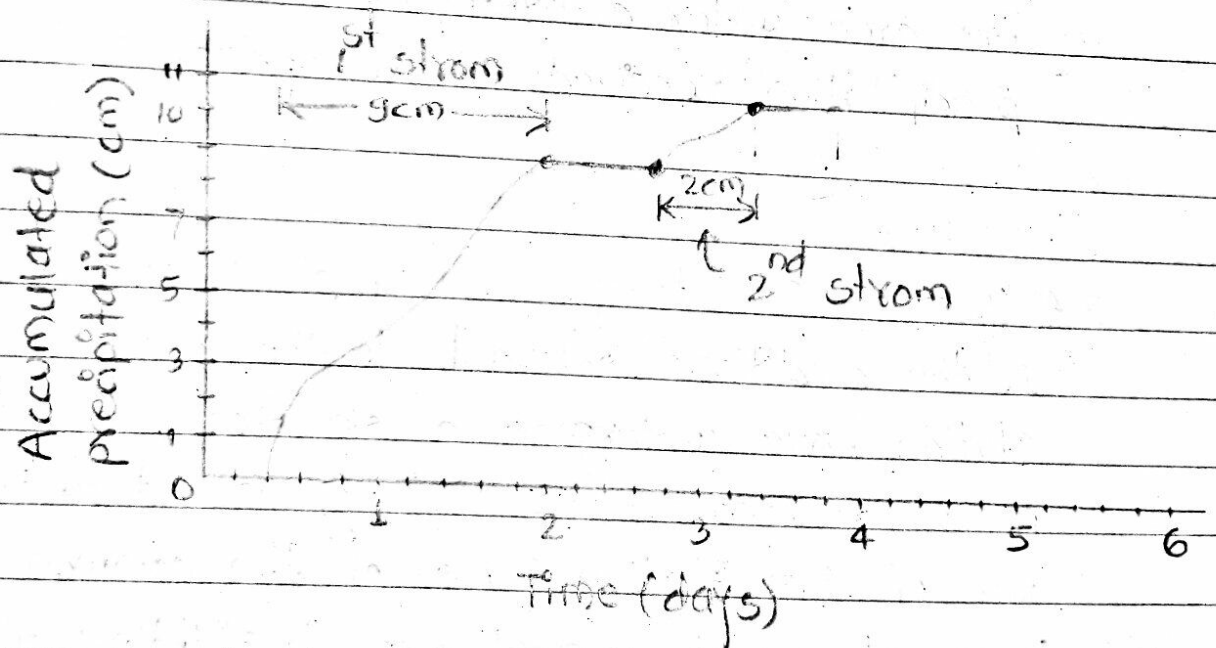


Fig:- Mass Curve of Rainfall

b) Hyetograph

A hyetograph is a plot of the intensity of rainfall against the time interval. The hyetograph is derived from the mass curve & is usually represented as a bar chart, as shown in figure below.

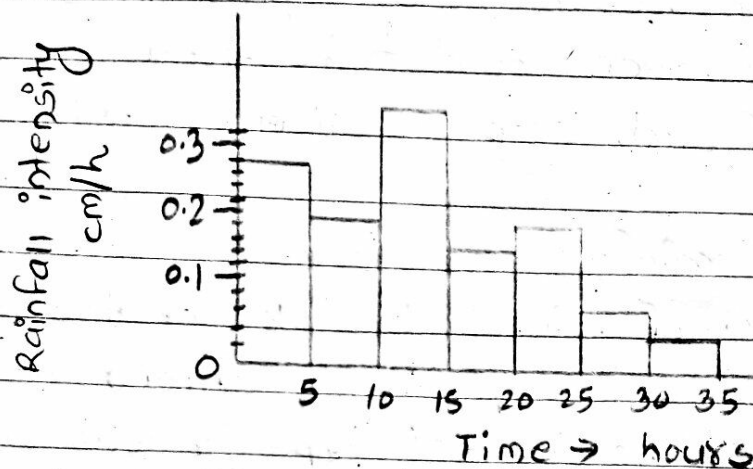


Fig. Hyetograph of a storm

The area under a hyetograph represents the total precipitation received in the period.

3.4) Test of Consistency of Rainfall Records

If the conditions relevant to the recording of a raingauge station have undergone a significant change during the period of record, inconsistency would arise in the rainfall data of that station. Some of the common causes for inconsistency of record are: i) shifting of a raingauge station to a new location. ii) the neighbourhood of the

station undergoing a marked change. iii) change in the ecosystem due to calamities, such as forest fires, land slides. iv) occurrence of observational error from a certain date. The checking for inconsistency of a record is done by the double-mass curve technique. This technique is based on the principle that when each recorded data comes from the same parent population, they are consistent.

A group of 5 to 10 base stations in the neighbourhood of the problem station X is selected. The data of the annual rainfall of the station X and also the average rainfall of the group of base stations covering a long period is arranged in the reverse chronological order. The accumulated precipitation of the station X (i.e. ΣP_x) and the accumulated values of the average of the group of base stations (i.e. ΣP_{av}) are calculated starting from the last record. Values of ΣP_x are plotted against ΣP_{av} for various consecutive time periods. A decided break in the slope of the resulting plot indicates a change in the precipitation regime of station X. The precipitation values at station X beyond the period of change of regime is corrected by using the relation

$$P_{cx} = P_x \cdot \frac{M_c}{M_a}$$

where, P_{cx} = corrected pptⁿ at any time period t_i at st. X
 P_x = original recorded pptⁿ at time period t_i at st. X
 M_c = corrected slope of double mass curve
 M_a = original slope of double mass-curve.

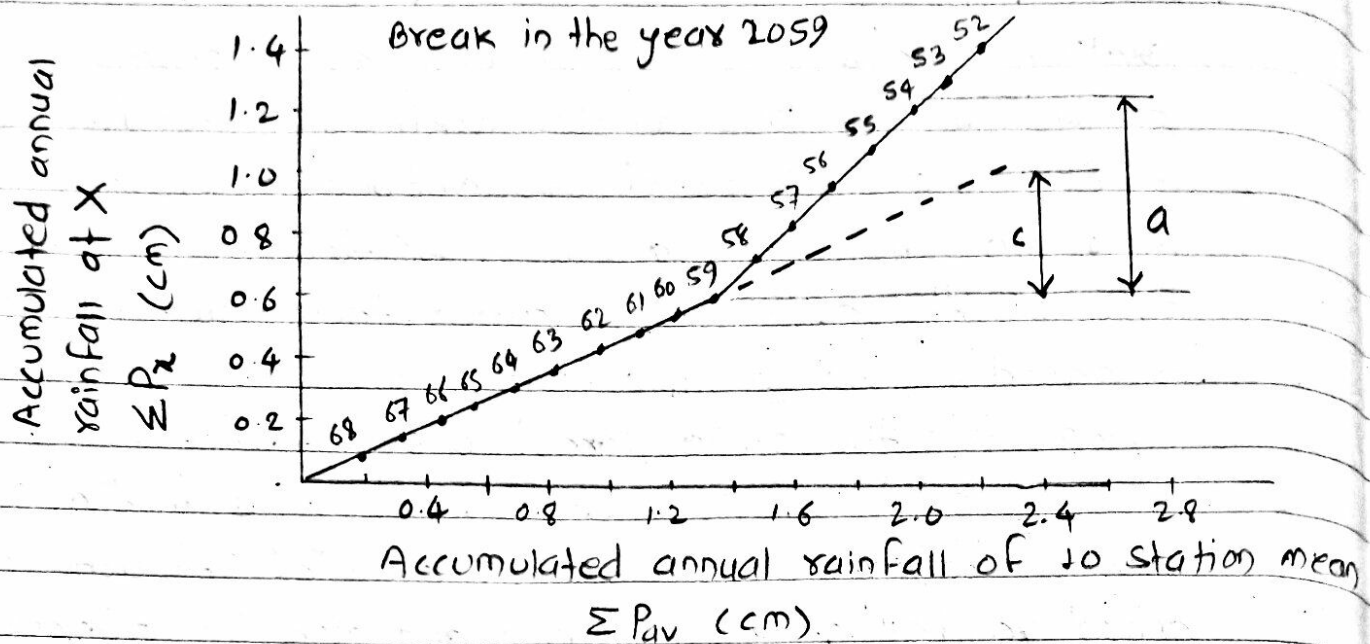


Fig. Double mass curve

In this way the older records are brought to the new regime of the station. It is apparent that the more homogeneous the base station records are, the more accurate will be the corrected values at station X. A change in the slope is normally taken as significant only where it persists for more than five years. The double mass curve is also helpful in checking systematic arithmetical errors in transferring rainfall data from one record to another.

3.6) Estimation of Missing Rainfall Data

Preparation of data

Before using the rainfall records of a station, it is

necessary to first check the data for continuity & consistency. The continuity of a record may be broken with missing data due to many reasons such as damage or fault in a raingauge during a period. The missing data can be estimated by using the data of the neighbouring stations.

Estimation of missing data

Given the annual precipitation values $P_1, P_2, P_3, \dots, P_m$ at neighbouring M stations $1, 2, 3, \dots, M$ respectively, it is required to find the missing annual precipitation P_x at a station X not included in the above M stations. Further, the normal annual precipitations $N_1, N_2, \dots, N_i, \dots$ at each of the above $(M+1)$ stations including station X are known.

If the normal annual precipitations at various stations are within about $\pm 10\%$ of the normal annual precipitation at station X , then a simple arithmetic average procedure is followed to estimate P_x . Thus,

$$P_x = \frac{1}{M} [P_1 + P_2 + P_3 + \dots + P_m]$$

If the normal precipitations vary considerably, then P_x is estimated by weighting the precipitation at the various stations by the ratios of normal annual precipitations. This method, known as the normal ratio method, gives P_x as

$$P_x = \frac{N_x}{M} \left[\frac{P_1}{N_1} + \frac{P_2}{N_2} + \dots + \frac{P_m}{N_m} \right]$$

2.14) The mass curve of rainfall in a storm of total duration 90 minutes is given below. a) draw the hyetograph of the storm at 20 minutes time step. b) Plot the Maximum intensity-duration curve for this storm. c) draw the mass curve of given time vs cumulative rainfall d) Plot the Maximum depth-duration curve for the storm.

Time (min)	0	10	20	30	40	50	60	70	80	90
cumulative										
rainfall (mm)	0	2.1	6.3	14.5	21.7	27.9	33.0	35.1	36.2	37.0

Solⁿ Given.

time (min)	cumulative rainfall (mm)	Incremental rainfall (Δp) (mm)									
		10	20	30	40	50	60	70	80	90	
10	2.1	2.1	-	-	-	-	-	-	-	-	
20	6.3	4.2	6.3	-	-	-	-	-	-	-	
30	14.5	8.2	12.4	14.5	-	-	-	-	-	-	
40	21.7	7.2	15.4	19.6	21.7	-	-	-	-	-	
50	27.9	6.2	13.4	21.6	25.8	27.9	-	-	-	-	
60	33.0	5.1	11.3	18.5	26.7	30.9	33.0	-	-	-	
70	35.1	2.1	7.2	13.4	20.6	28.8	33.0	35.1	-	-	
80	36.2	1.1	3.2	8.3	14.5	21.7	29.9	34.1	36.2	-	
90	37.0	0.8	1.9	4.0	9.1	15.3	22.5	30.7	34.9	37.0	

Also, the time, maximum depth & maximum intensity table is given below:

Numerical $i = \frac{\Delta P}{\Delta t} = \frac{8.2}{10} \times 60 = \frac{\text{mm}}{\text{hr}}$ Page:- 55/54.

time	10	20	30	40	50	60	70	80	90
max depth (mm)	8.2	15.4	21.6	26.7	30.9	33.0 33.0	35.1 33.0	36.2	37.0
max intensity $i = \frac{\Delta P}{\Delta t}$ (mm/hr)	49.2	46.2	43.2	40.05	37.08	33.0	30.09	27.15	24.67

a) \Rightarrow The hyetograph of the storm at 20 minutes time step.

time (min)	cumulative rainfall (mm)	Incremental rainfall in 20 minutes ΔP	$i = \frac{\Delta P}{\Delta t}$ (mm/hr)
10	2.1	—	—
20	6.3	$6.3 - 0 = 6.3$	$0.315 \times 60 = 18.9$
30	14.5	$14.5 - 2.1 = 12.4$	$0.62 \times 60 = 37.2$
40	21.7	$21.7 - 6.3 = 15.4$	$0.77 \times 60 = 46.2$
50	27.9	$27.9 - 14.5 = 13.4$	$0.67 \times 60 = 40.2$
60	33.0	$33.0 - 21.7 = 11.3$	$0.565 \times 60 = 33.9$
70	35.1	$35.1 - 27.9 = 7.2$	$0.36 \times 60 = 21.6$
80	36.2	$36.2 - 33.0 = 3.2$	$0.16 \times 60 = 9.6$
90	37.0	$37.0 - 35.1 = 1.9$	$0.095 \times 60 = 5.7$

In graph.

b) In graph c) In graph d) In graph.

2.12) Following are the data of a storm as recorded in a self recording rain gauge at a station.

Time from beginning of storm (min)	10	20	30	40	50	60	70	80	90
cumulative rainfall (mm)	19	41	48	68	91	124	152	160	166

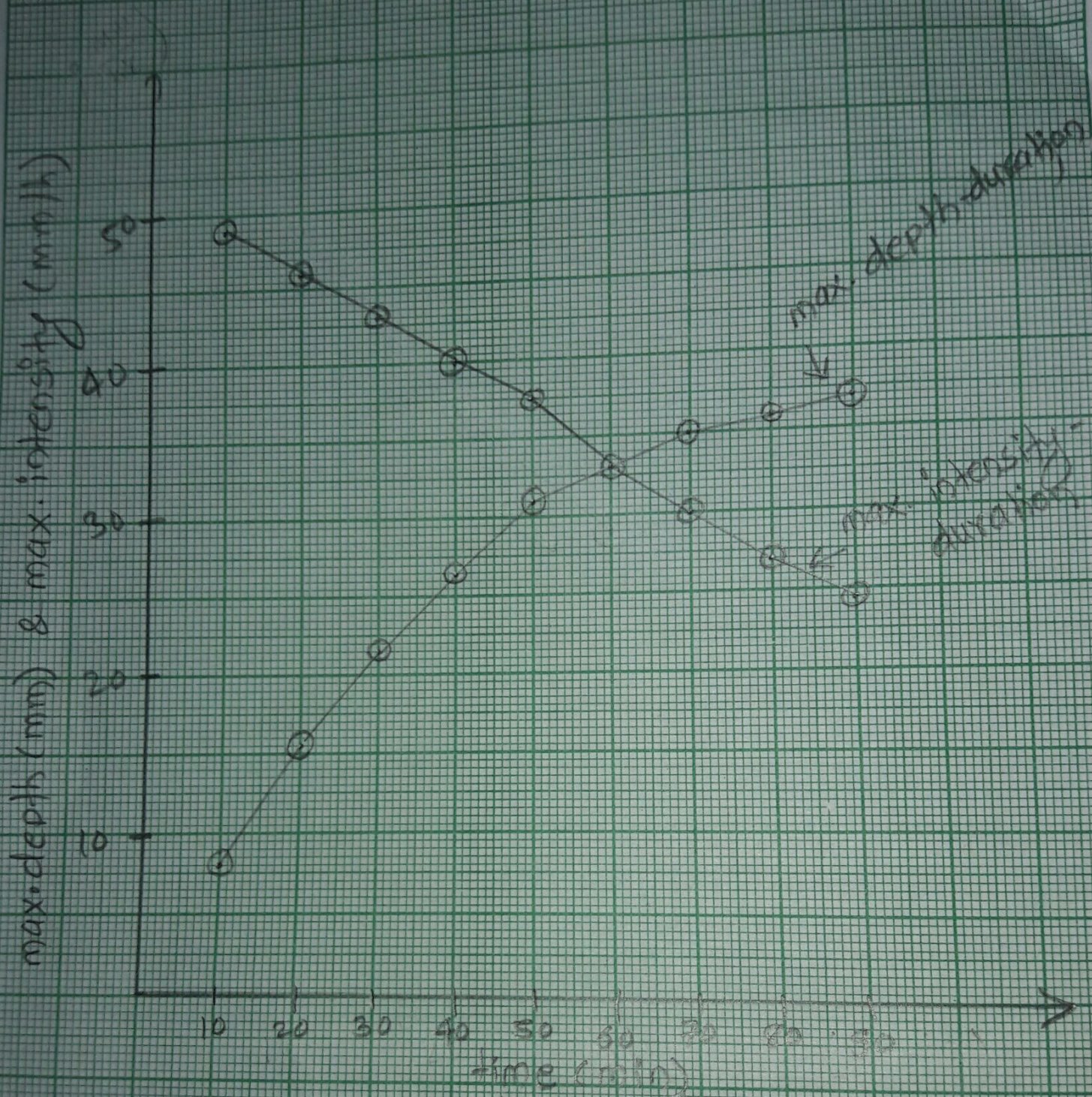


Fig. max. intensity duration curve & max. depth duration curve

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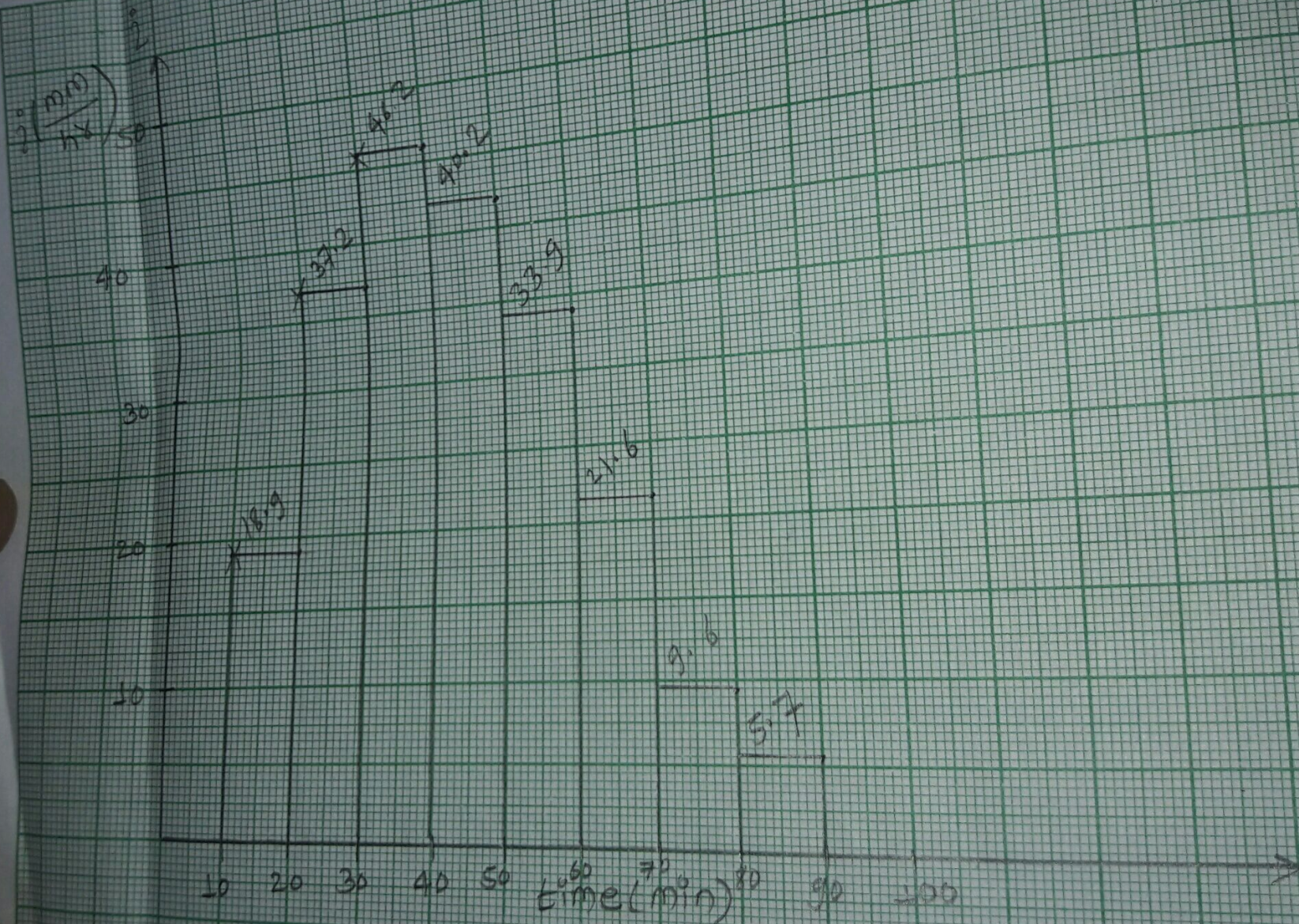


Fig. Hyetograph of the storm

Soln Given,

time (min)	cumulative rainfall (mm)	Incremental rainfall ΔP (mm)								
		10	20	30	40	50	60	70	80	90
10	19	19	-	-	-	-	-	-	-	-
20	41	22	41	-	-	-	-	-	-	-
30	48	7	29	48	-	-	-	-	-	-
40	68	20	27	49	68	-	-	-	-	-
50	91	23	43	50	72	91	-	-	-	-
60	124	(33)	56	76	83	105	124	-	-	-
70	152	28	(61)	(84)	(104)	111	(133)	(152)	-	-
80	160	8	36	69	92	(112)	119	141	(160)	-
90	166	6	14	42	65	98	118	125	147	(166)

a) \Rightarrow For the hyetograph of the storm,

time (min)	cumulative rainfall (mm)	Incremental rainfall In 10 min,	$i = \frac{\Delta P}{\Delta t}$ (mm/hr)
10	19	$19 - 0 = 19$	$\frac{19}{10} \times 60 = 114$
20	41	$41 - 19 = 22$	$\frac{22}{10} \times 60 = 132$
30	48	$48 - 41 = 07$	$\frac{7}{10} \times 60 = 42$
40	68	$68 - 48 = 20$	$\frac{20}{10} \times 60 = 120$
50	91	$91 - 68 = 23$	$\frac{23}{10} \times 60 = 138$
60	124	$124 - 91 = 33$	$\frac{33}{10} \times 60 = 198$
70	152	$152 - 124 = 28$	$\frac{28}{10} \times 60 = 168$
80	160	$160 - 152 = 08$	$\frac{8}{10} \times 60 = 48$
90	166	$166 - 160 = 06$	$\frac{6}{10} \times 60 = 36$

$$\frac{33}{10} \times 60 = 198$$

time (min)	10	20	30	40	50	60	70	80	90
depth (mm) ^{max.} (op)	33	61	84	104	112	133	152	160	166
max. intensity $\frac{dp}{dt}$ ($\frac{mm}{hr}$)	198	183	168	156	134.4	133	130.29	120	110.67

a) In graph b) In graph

$\dot{V} \left(\frac{\text{mm}}{\text{hr}} \right)$

$$20 \text{ SD} = 302$$

$$10 \text{ SD} = 10 \text{ min}$$

$$\frac{2}{3} \times 114$$

210

180

150

120

90

60

30

0

10

20

30

40

50

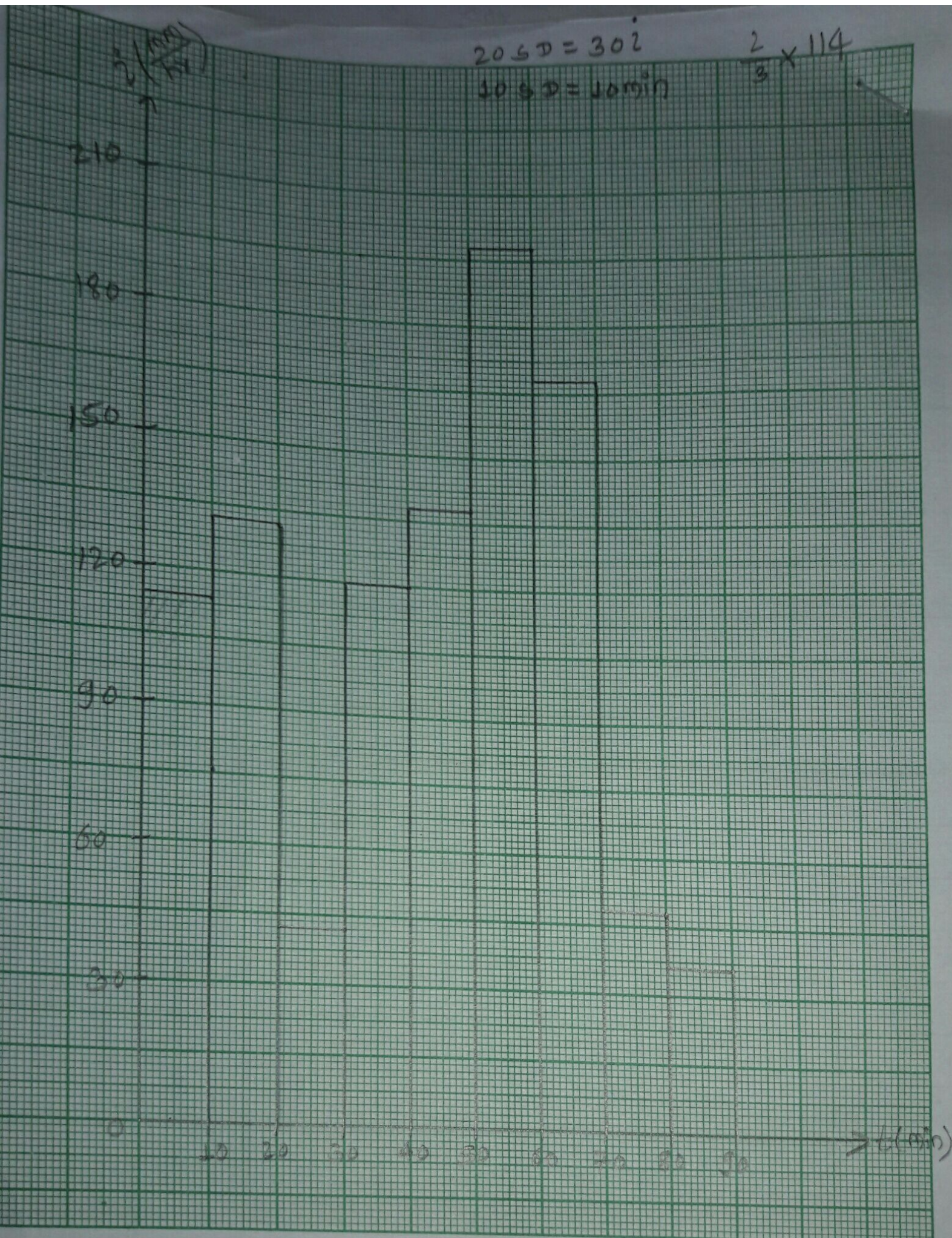
60

70

80

90

$t \text{ (min)}$



20 SD = 30°

10 SD = 10 min

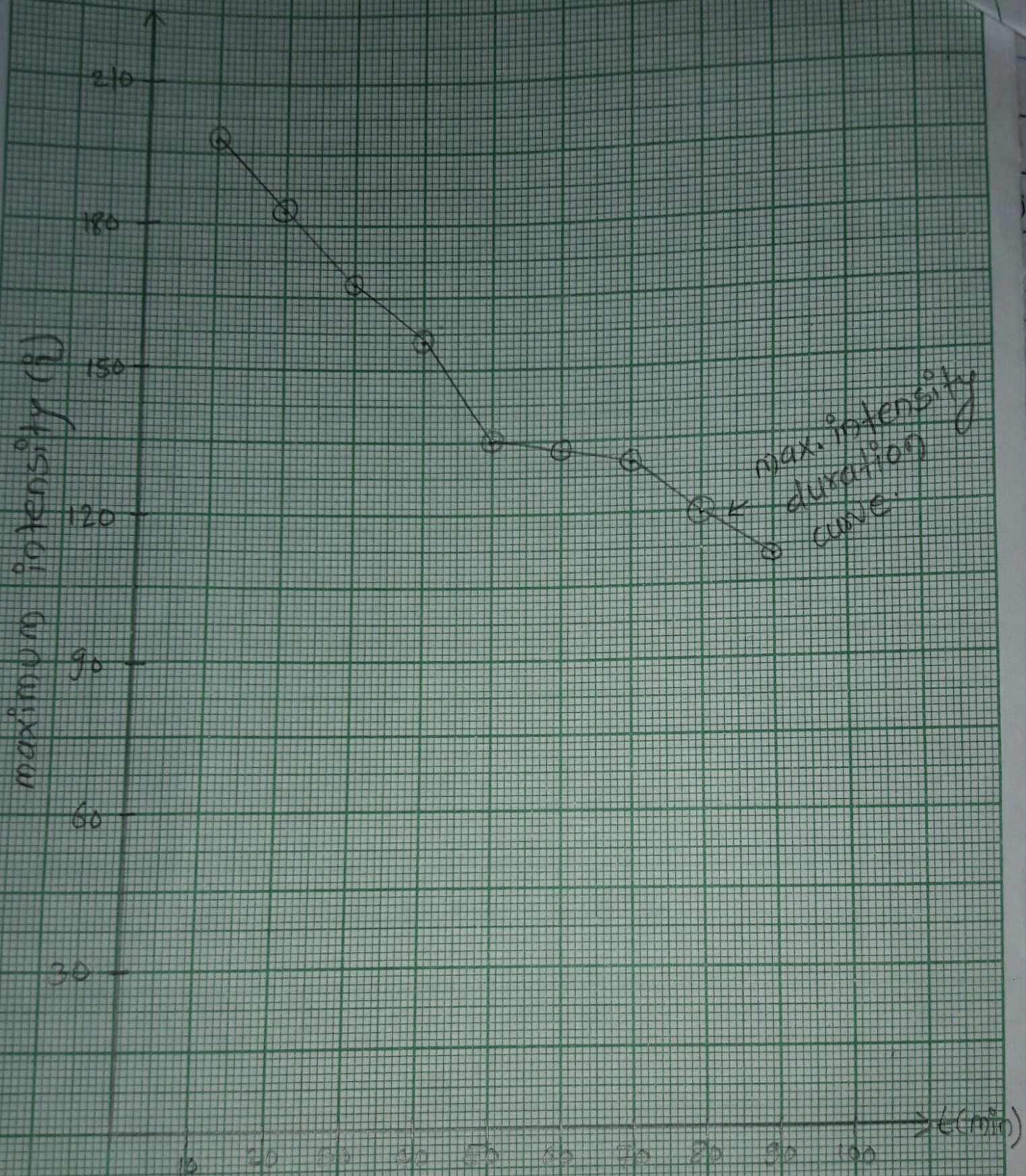


Fig. - maximum intensity duration curve

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chapter :- 6 (Runoff & stream flow)

6.1) Definition & components of Runoff

* Definition

Runoff is defined as the excessive rainfall which flow from the ground surface, sub-surface & ground water flow to the stream.

* components

- i) surface runoff / overland flow
- ii) subsurface runoff / interflow
- iii) groundwater flow / base flow.

Total runoff from the area = surface flow + ^{direct runoff} inter flow + ground water flow
→ Base flow.

6.2) Factors Affecting Runoff.

Factor affecting runoff from a catchment is listed as below.

i) Basin characteristics

- a) Shape
- b) Size
- c) Slope
- d) Nature of valley
- e) Elevation
- f) drainage density

- 2) Infiltration characteristics
- a) land used & cover
 - b) soil type & geological conditions.
 - c) lakes & other storage

- 3) channel characteristics
- a) cross-section
 - b) Roughness
 - c) storage capacity

The above all factors is physiographic factors and also climatic factors are affected runoff from a catchment.

- 1) climatic factors

- a) storm characteristics:

precipitation, intensity, duration, magnitude & movement of storm.

- b) Initial loss

- c) Evapotranspiration.

6.3) Estimation of Runoff

The runoff from rainfall may be estimated by the following methods.

- a) Empirical formulae, curves & tables.

- b) Infiltration method

- c) Rational method

- d) overland flow hydrograph

e) Unit hydrograph method

6)
The above methods are discussed as follows:-

a) Empirical formulae, curves & tables

Several empirical formulae, curves & tables relating to the rainfall & runoff have been developed as follows.

Usually, $R = aP + b$

Sometimes, $R = aP^n$

where,

R = runoff, P = rainfall, a , b & n are constants

b) Infiltration method

By deducting the infiltration loss i.e. the area under the infiltration curve, from the total precipitation, runoff can be estimated.

c) Rational method

A rational approach is to obtain the yield of a catchment by summing a suitable runoff coefficient.
 $\text{Yield} = CAP$

where, A = area of catchment

P = precipitation

C = runoff coefficient

The value of the runoff coefficient C varies depending upon the soil type, vegetation geology, etc.

d) Overland Flow Hydrograph

Overland flow occurs as a thin sheet of water over the ground surface (soon after a storm starts), joins a stream channel, and then flows in the channel to the concentration point. Overland flow is relatively slow & is the dominant type of flow in the case of very small areas.

Overland flow is essentially a uniform flow over the surface. The Reynold number,

$$Re = \frac{vd}{\nu} = \frac{q}{\nu}$$

where, v = velocity of flow

d = uniform depth of flow

ν = kinematic viscosity of water

q = discharge per unit width.

Experiments indicate that the overland flow can be assumed to be laminar if $Re \leq 1000$ & turbulent if $Re > 1000$.

e) Unit Hydrograph method

The hydrograph of direct surface discharge measured at the outlet of drainage area, which produces a unit depth of direct runoff resulting from a unit storm of specified duration is called a unit hydrograph of that duration. The area under the hydrograph represents a direct runoff of 1 cm.

6.4) Rainfall-Runoff Relationship.

The relationship between rainfall in a period and the corresponding runoff is quite complex. So, one of the most common methods is to correlate seasonal or annual measured runoff value (R) with corresponding rainfall (P) values. A commonly adopted method is to fit a linear regression line between R & P and to accept the result if the correlation coefficient is nearly unity. The equation of the straight line regression between runoff R & rainfall P is,

$$R = aP + b \quad \text{--- ①}$$

& the values of the coefficient a & b are given by

$$a = \frac{N(\sum PR) - (\sum P)(\sum R)}{N(\sum P^2) - (\sum P)^2}$$

$$\& b = \frac{\sum R - a(\sum P)}{N}$$

in which N = number of observation sets R & P .

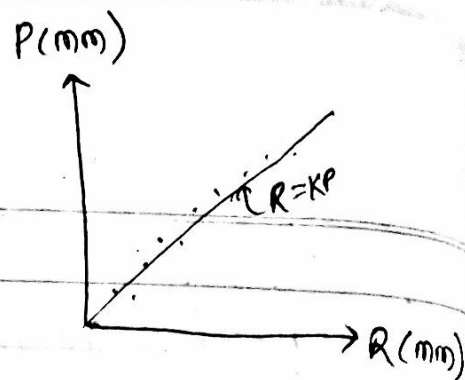
The coefficient of correlation r can be calculated as,

$$r = \frac{N(\sum PR) - (\sum P)(\sum R)}{\sqrt{N(\sum P^2) - (\sum P)^2} \sqrt{N(\sum R^2) - (\sum R)^2}}$$

The value of r lies between 0 & 1. as R can have only positive correlation with P . The value of $0.6 < r < 1$ indicates good correlation. Further, it should be noted that $R \geq 0$.

For large catchments, sometimes it is found advantageous to have exponential relationship as,

Total = municipality + usx 719



$$R = \beta P^m$$

where β and m are constants,

$$\text{or, } \ln R = \ln \beta + m \ln P.$$

& β and m are determined as a & b of previous method.

6.5) Stream Gauging : types & site selection.

* Selection of site for a stream gauging station.

Stream gauging is defined as the location at which the river discharges are recorded and the discharge measurements are carried out & prepared rating curve is called stream gauging.

The following factors have to be considered in selecting a site for a stream gauging station.

- The section should be straight and uniform for a length of about 10 to 20 times the width of stream.
- The supports of gauge should be rigid & immovable.
- The bed & banks should be free from vegetal growth, boulders or other obstructions.
- At the selected discharge site, water should flow in a single channel. It should not overflow the banks.
- The channel should have regular cross-section.
- Velocities should be neither too high nor too low, generally in the range of 0.3 - 1.2 m/s.
- The stream gauging station should be easily accessible.

6.6) Rating curves.

The relationship between discharge (Q) & stage (H) is known as rating curves. Continuous measurement of discharge is not feasible so, rating curves are used to convert the measured stage (H) into discharge (Q).

Equation of rating curve,

$$Q = C_r (H - a)^\beta \quad \text{--- ①}$$

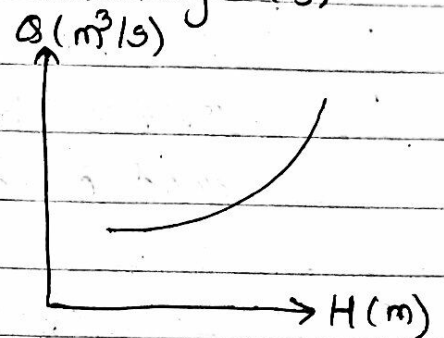
where,

Q = stream discharge

H = gauge height (stage)

a = a constant which represent the gauge reading corresponding to zero discharge.

C_r, β = rating curve constants.



Converting this equation to logarithmic form gives simple linear equation, which is then easy to use for further analysis.

$$Q = C_r (H - a)^\beta$$

$$\ln Q = \beta \ln(H - a) + \ln C_r$$

comparing equation with $y = \beta x + b$

$$\text{Since } y = \ln Q$$

$$x = \ln(H - a)$$

$$b = \ln C_r$$

$$\therefore \beta = \frac{N(\sum xy) - (\sum x)(\sum y)}{N(\sum x^2) - (\sum x)^2}, \quad b = \frac{\sum y - \beta(\sum x)}{N}$$

And, correlation coefficient is given by,

$$r = \frac{N(\sum xy) - (\sum x)(\sum y)}{\sqrt{N(\sum x^2) - (\sum x)^2} \sqrt{N(\sum y^2) - (\sum y)^2}}$$

Here,

r reflects the extend of linear relationship between the two data sets. For a perfect correlation $r = \pm 1$. If r is between 0.6 & 1.0, it is generally taken as good correlation.

* Uses of Rating Curves

It is used for estimating the discharge 'Q' of the stream for a given gauge reading.

6.7) Stream flow measurement using direct & indirect methods.

The methods for determining the stream flow are

a) Direct determination of Stream flow

= i) Area velocity method

ii) Dilution technique (salt concentration of chemical method)

iii) Electromagnetic method

iv) Ultrasonic method.

b) Indirect method of stream flow

i) Hydraulic structures such as weirs, notches, gates, etc.

ii) Slope area method (to estimate peak flood where gauging station exist).

* Area-velocity method.

This method of discharge measurement consists essentially of measuring the area of cross-section of the river at a selected section called the gauging site & measuring the velocity of flow through the cross-sectional area.

The gauging site should have the following criteria

- a) The stream should have a well-defined cross-section which does not change in various seasons.
- b) It should be easily accessible all through the year.
- c) The site should be in a straight, stable reach, etc.

Velocity area method is based on the continuity equation

$$Q = AV.$$

where,

Q = discharge of the stream (m^3/s)

A = cross-sectional area (m^2)

V = mean velocity in the c/s of stream (m/s).

Discharge is determined by measuring cross-sectional area and velocity. The velocity is measured by means of floats or by using a current meter,

$$\text{i.e. } V = a N_s + b \quad \text{--- (1)}$$

where,

N_s = revolution/sec.

a & b are meter constant

consider the cross-section of the river in which $N-1$

verticals are drawn. The velocity averaged over the vertical at each section is known. considering the total area to be divided into $N-1$ segments, the total discharge is calculated by the method of mid-sections as follows,

$$Q = \sum_{i=1}^{N-1} \Delta Q_i$$

where,

$\Delta Q_i =$ discharge in the i^{th} segment

$= (\text{depth at } i^{\text{th}} \text{ segment}) \times (\frac{1}{2} \text{ width to the left} + \frac{1}{2} \text{ width to the right}) \times (\text{average velocity at } i^{\text{th}} \text{ vertical})$

$$\Delta Q_i = y_i \times \left(\frac{w_i}{2} + \frac{w_{i+1}}{2} \right) \times V_i \quad \text{for } i=2 \text{ to } (N-2)$$

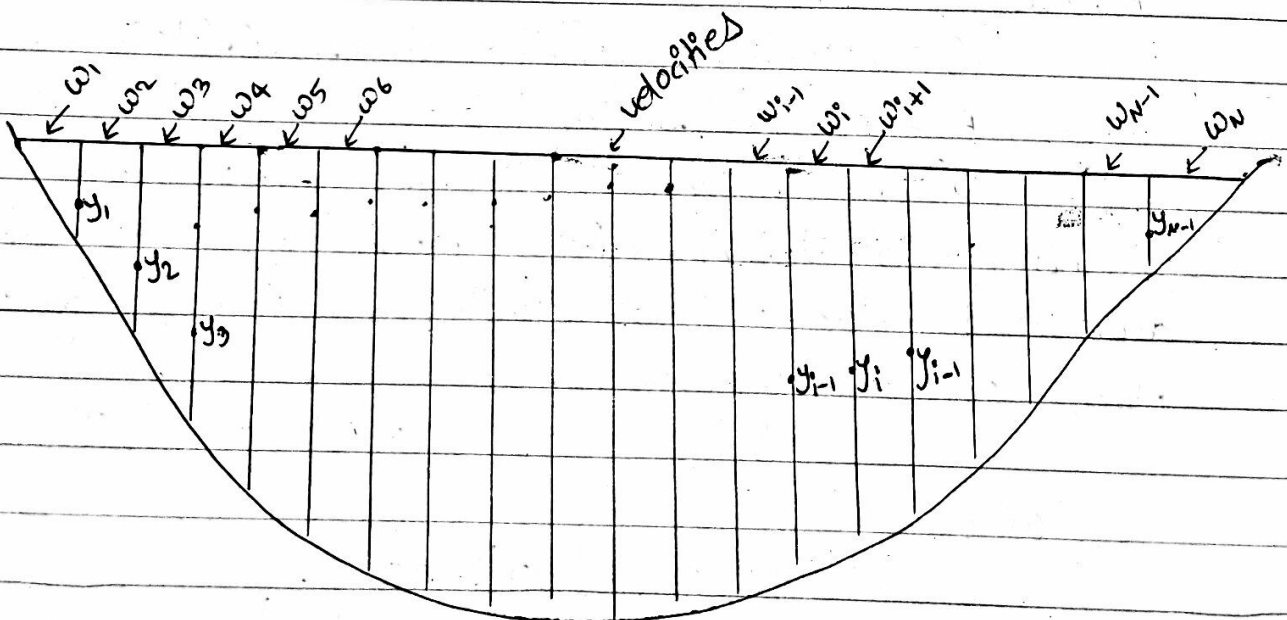


fig. stream section for Area-velocity method

For the first & last sections, the segments are taken to have triangular areas & area calculated as,

$$\Delta A_1 = \bar{w}_1 y_1$$

$$\text{where } \bar{w}_1 = \frac{(w_1 + \frac{w_2}{2})^2}{2w_1}$$

$$\& \Delta A_{N-1} = \bar{w}_{N-1} y_{N-1}$$

$$\text{where } \bar{w}_{N-1} = \frac{(w_N + \frac{w_{N-1}}{2})^2}{2w_N}$$

to get

$$\Delta S_1 = \bar{v}_1 \cdot \Delta A_1 \& \Delta S_{N-1} = \bar{v}_{N-1} \cdot \Delta A_{N-1}$$

* Slope area method

This is indirect method of discharge measurement. In this equation method, Manning's eqⁿ & Bernoulli's equation are used to estimate the discharge for high floods based on the previous flood marks.

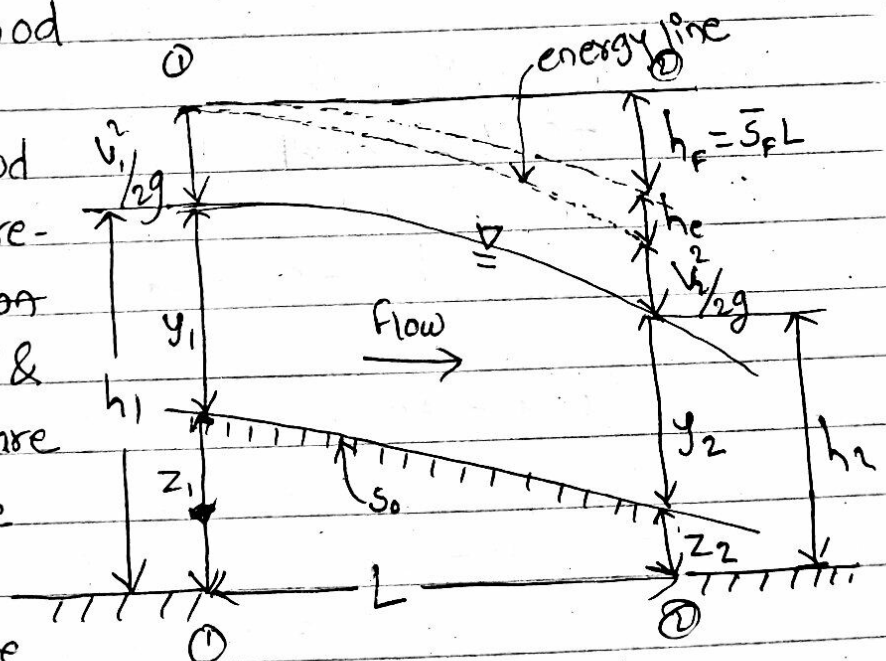


Fig. slope area method

Two stations along a river reach are selected. The

cross-sectional area of each station and the longitudinal profile between the stations is measured. knowing the water-surface elevations at the two sections, it is required to estimate the discharge. Applying the energy equation (Bernoulli's equation) to section 1 & 2.

$$z_1 + y_1 + \frac{v_1^2}{2g} = z_2 + y_2 + \frac{v_2^2}{2g} + h_L$$

where h_L = head loss in the reach. The head loss can be made up of two parts i) frictional loss h_f & ii) eddy loss h_e . Denoting $z + y = h$ = water surface elevation above the datum,

$$h_1 + \frac{v_1^2}{2g} = h_2 + \frac{v_2^2}{2g} + h_e + h_f$$

$$\text{or, } h_f = (h_1 - h_2) + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} \right) - h_e$$

a) for uniform flow

IF L = length of reach, by Manning's formula for uniform flow,

$$\frac{h_f}{L} = S_f = \text{energy slope} = \frac{Q^2}{K^2}$$

$$\& v = \frac{1}{n} R^{2/3} S^{1/2} \& Q = \frac{A}{n} R^{2/3} S^{1/2}$$

where,

$$K = \text{conveyance of the channel} = \frac{1}{n} A R^{2/3}$$

[$S_o = S_f = S_w$]

b) for non-uniform flow

$$\frac{h_f}{L} = \bar{S}_f = \frac{Q^2}{K^2}$$

where $K = \sqrt{K_1 K_2}$; $K_1 = \frac{1}{n_1} A_1 R_1^{2/3}$

$$K_2 = \frac{1}{n_2} A_2 R_2^{2/3}$$

The eddy loss h_e is estimated as,

$$h_e = K_e \left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right]$$

where K_e = eddy-loss coefficient.

* Procedure to compute peak discharge by using slope-area method.

- 1) Compute cross-sectional area, wetted perimeter, hydraulic radius, conveyance K_1 & K_2 at section 1 & 2, compute K using $K = \sqrt{K_1 K_2}$
- 2) For first iteration, assume $V_1 = V_2$, this leads to $h_f = h_1 - h_2$
- 3) compute Q using equation $Q = K \sqrt{S_f} = K \sqrt{h_f/L}$
- 4) compute $V_1 = Q/A_1$ & $V_2 = Q/A_2$
- 5) Now calculate a refined value of h_f by using equation $h_f = (h_1 - h_2) + (V_1^2/2g - V_2^2/2g) - h_e$
- 6) Taking refined value of h_f for next iteration & repeat steps 3 to 5 until the difference between two successive values of h_f is negligible.
- 7) compute Q using final value of h_f .

6.8) Use and calibration of current meter.

The most commonly used instrument in hydrometry to measure the velocity at a point in the flow cross-section is the current meter. It consists essentially of a rotating element which rotates due to the reaction of the stream current with an angular velocity proportional to the stream velocity. A current meter is so designed that its rotation speed varies linearly with the stream velocity V at the location of the instrument.

A typical relationship is,

$$V = a N_s + b \quad \text{--- ①}$$

where,

V = stream velocity at the instrument location (m/s).

N_s = revolution per second of the meter

a & b = constants of the meter.

* calibration of current meter

The relationship between the stream velocity and revolutions per second of the meter as in equation ① is called the calibration equation. The calibration equation is unique to each instrument & is determined by towing the instrument in a special tank. A towing tank is a long channel containing still water with arrangements for moving a carriage longitudinally over its surface at constant speed. The instrument to be calibrated is mounted on the carriage with the rotating element immersed to a

specified depth in the water body in the tank. The carriage is then towed at a predetermined constant speed (U) and the corresponding average value of revolutions per second (N_s) of the instrument is determined. This experiment is repeated over the complete range of velocities and a best fit linear relation in the form of equation ① is obtained. For the point of view of accuracy it is advised to check the instrument calibration once in a while and whenever there is a suspicion that the instrument is damaged due to bad handling or accident.

chapter :- 4 (Infiltration & Percolation).

4.3) Infiltration, Infiltration capacity.

* Infiltration.

The phenomenon of inflow of water into the ground through the soil surface is called infiltration.

* Infiltration capacity

The maximum rate at which a given soil at a given time can absorb water is defined as the infiltration capacity. It depends on time & decrease with respect to duration of time of rainfall. It is designated as f_p and is expressed in units of cm/hr.

The actual rate of infiltration F can be expressed as,

$$F = f_p \text{ when } i \geq f_p$$

$$\& F = i \text{ when } i < f_p$$

where i = intensity of rainfall.

4.4) Infiltration indices

Average rate of infiltration is called infiltration index. Two types of indices are commonly used. They are

a) ϕ -index

b) ω -index

a) ϕ -index

The ϕ -index is the average rainfall above which the rainfall volume is equal to the runoff volume. The ϕ index is

derived from the rainfall hyetograph with the knowledge of the resulting runoff volume. The initial loss is also considered as infiltration.

The ϕ -value is found by treating it as a constant infiltration capacity. If the rainfall intensity is less than ϕ , then the infiltration rate is equal to the rainfall intensity; however, if the rainfall intensity is larger than ϕ , the difference between the rainfall and infiltration in an interval of time represents the runoff volume as shown in figure above.

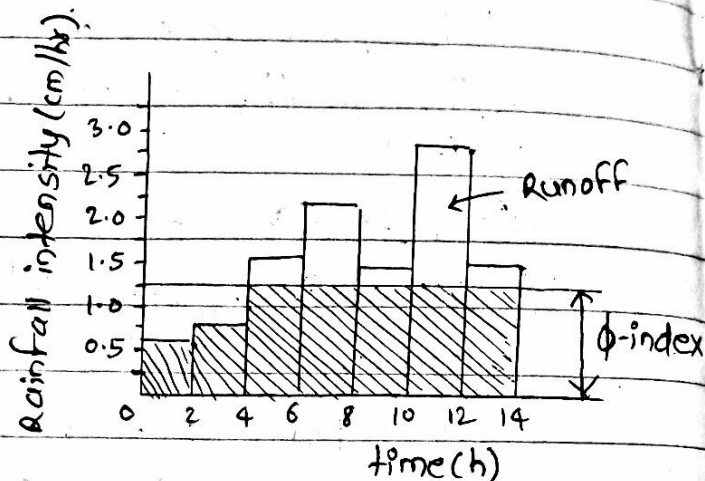


Fig. ϕ -index

* Procedure for calculation of ϕ -index

Consider a rainfall hyetograph of event duration D hours having N pulses of time interval Δt such that

$$N \cdot \Delta t = D \quad \text{in above fig. } N=7$$

Let I_i be the intensity of rainfall in i th pulse & R_d = total direct runoff.

$$\text{Total Rainfall} = \sum_{i=1}^N I_i \cdot \Delta t$$

If ϕ is ϕ -index, then $P - \phi \cdot t_e = R_d$

where t_e = duration of rainfall excess.

If the rainfall hyetograph and total runoff depth R_d are given, the ϕ -index of the storm can be determined by trial & error procedure as given below.

1) Assume that out of given N pulses, M number of pulses have rainfall excess. (Note that $M \leq N$). Select M numbers of pulses in decreasing order of rainfall intensity I_i .

2) Find the value of ϕ that satisfies the relation,

$$R_d = \sum_{i=1}^M (I_i - \phi) \Delta t \quad \phi = \frac{P_e - Q}{t_e}$$

where,

P_e = total precipitation during which runoff take

Q = total amount of runoff,

t_e = effective time at which runoff occurs.

3) Find the value of Q using the relation,

$$Q = \sum (I_i - \phi) \Delta t$$

4) If Q is calculated Q is equal to the given Q then the ϕ is correct. If Q is not equal then a value of P_e & t_e is calculated.

5) By using the value of P_e & t_e , value of ϕ is calculated

6) By using ϕ , Q is calculated

7) If value of calculated Q is equal to given value of Q , then Q is correct if not repeat from steps 4.

b) w -index

In an attempt to refine the ϕ -index the initial losses are separated from the total abstractions and an average

value of infiltration rate, called w -index, is defined as,

$$W = \frac{P - R - I_a}{t_e}$$

where, P = total storm precipitation (cm)

R = total storm runoff (cm)

I_a = Initial losses (cm)

t_e = duration of the rainfall excess, i.e. the total in which the rainfall intensity is greater than W (in hours) and

W = defined average rate of infiltration (cm).

Since I_a rates are difficult to obtain, the accurate estimation of W -index is rather difficult.

The minimum value of the W -index obtained under very wet soil conditions, representing the constant minimum rate of infiltration of the catchment, is known as W_{min} . It is to be noted that both the ϕ -index & W -index vary from storm to storm.

* computation of w -index

To compute W -index from a given storm hyetograph with known values of I_a & the runoff R , the following procedure is followed.

- 1) Deduct the initial loss I_a from the storm hyetograph.
- 2) Using the resulting hyetograph pulse diagram & following the procedure as ϕ -index, w -index is determined.

Thus the procedure is exactly same as in the determination of ϕ -index except for the fact that the storm hyetograph is approximately modified by deducting I_a .

4.6) Horton's equation.

Horton expressed the decay of infiltration capacity with time as an exponential decay given by,

$$f_p = f_c + (f_0 - f_c) e^{-k_h t} \quad \text{for } 0 \leq t \leq t_c$$

where,

f_p = infiltration capacity at any time t from the start of the rainfall

f_0 = initial infiltration capacity at $t=0$.

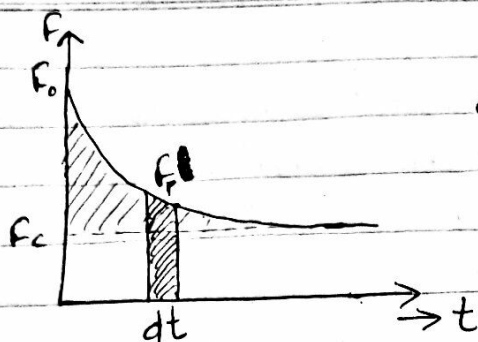
f_c = final steady state infiltration capacity occurring at $t=t_c$. Also, f_c is sometimes known as constant rate or ultimate infiltration capacity.

k_h = Horton's decay constant coefficient which depends upon soil characteristics & vegetation cover.

अर्थ
definition,

According to Horton, infiltration begins at some rate f_0 & exponentially decreases until it reaches a constant value f_c .

$$f_p = f_c + (f_0 - f_c) e^{-k_h t}$$



depth of infiltrated water within time dt .

$$dF = f dt$$

Total infiltrated water depth

$$F = \int_0^t f_p dt = \int_0^t (f_c + (f_0 - f_c)e^{-k_p t}) dt$$

$$\text{or, } F = f_c t + (f_0 - f_c) \left[\frac{e^{-k_p t}}{-k_p} \right]_0^t$$

$$\therefore F = f_c t + \left(\frac{f_0 - f_c}{k_p} \right) (1 - e^{-k_p t})$$

Average infiltration rate, $F_{avg} = F/t$

$$F_{avg} = f_c + \left(\frac{f_0 - f_c}{k_p t} \right) (1 - e^{-k_p t})$$

* Determination of Horton's constant

There are basically two methods for determining Horton's constant. They are,

- a) Graphical method in field.
- b) Least square method.

a) Graphical method in field.

Now, the Horton's equation is,

$$F_p = F_c + (F_0 - F_c)e^{-kt} \quad \text{--- ①}$$

$$\text{or, } (F_p - F_c) = (F_0 - F_c)e^{-kt}$$

taking natural ln on both sides,

$$\ln(F_p - F_c) = \ln(F_0 - F_c) - kt \quad \text{--- ②}$$

comparing above equation with $y = mx + c$, we get

$$y = \ln(F_p - F_c)$$

$$m = k$$

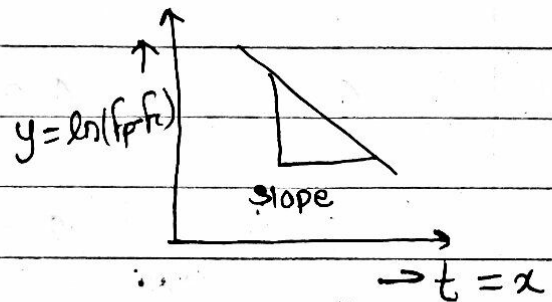
$$x = t$$

$$c = \ln(F_0 - F_c)$$

plot graph t Vs $\ln(F_p - F_c)$

calculate slope

$$k = \text{slope} (-m)$$



b) least square method

Since, the Huxton's equation is,

$$F_p = F_c + (F_0 - F_c)e^{-kt}$$

$$\text{or, } \ln(F_p - F_c) = \ln(F_0 - F_c) - kt \quad \text{--- ①}$$

comparing equation with $y = a + bx$, we get

$$y = \ln(F_p - F_c)$$

$$a = \ln(F_0 - F_c)$$

$$b = -k, x = t$$

$$\therefore b = \frac{n \sum xy - \sum x \sum y}{n \sum x^2 - (\sum x)^2}, a = \frac{\sum y - b \sum x}{n}$$

So, k is calculated

Numerical Problems

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Q.8) The following data are obtained from the current meter gauging of a stream, at a gauging station. compute the stream discharge

distance from one end of water surface (m)	depth of water, d (m)	Immersion of current meter below water surface		
		depph(m)	rev.	sec.
0	0	-	-	-
2	1.0	0.6	10	40
4	2.2	0.44	36	48
		1.76	20	50
6	4.0	0.80	40	57
		3.20	30	53
8	8.0	1.6	46	59
		6.4	33	57
10	4.2	0.84	33	51
		3.36	29	49
12	2.5	0.50	24	52
		2.00	29	53
14	1.2	0.72	16	48
16	0	-	-	-

i) mid section method

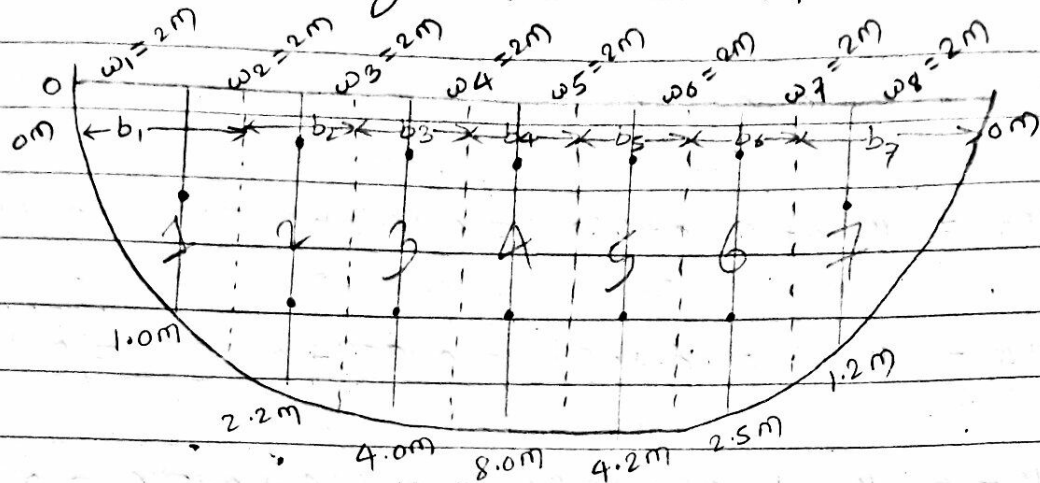
ii) mean section method

Rating equation of current meter: $V = 0.2N + 0.04$,
where $N = \text{rev/sec}$, $V = \text{velocity (m/s)}$.

Numerical Problems

Ex 12 i) By mid-section method

no. of segment = no. of depths = 7



Now, for the calculation of width,

considering the area of both sides of the bank,

$$b_1 = \frac{(w_1 + w_2/2)^2}{2w_1} = \frac{(2 + 2/2)^2}{2 \times 2} = 2.25m$$

$$b_2 = w_2/2 + w_3/2 = 2/2 + 2/2 = 2m$$

$$b_3 = w_3/2 + w_4/2 = 2/2 + 2/2 = 2m$$

$$b_4 = w_4/2 + w_5/2 = 2/2 + 2/2 = 2m$$

$$b_5 = w_5/2 + w_6/2 = 2/2 + 2/2 = 2m$$

$$b_6 = w_6/2 + w_7/2 = 2/2 + 2/2 = 2m$$

$$b_7 = \frac{(w_8 + w_7/2)^2}{2w_8} = \frac{(2 + 2/2)^2}{2 \times 2} = 2.25m$$

Numerical Problems

Then, average velocities of each section is given by,

$$V_1 = V \text{ at } 0.6d = 0.2 \times 10/40 + 0.04 = 0.09 \text{ m/s}$$

$$V_2 = V \text{ at } \frac{0.2d + 0.8d}{2} = \frac{0.19 + 0.12}{2} = 0.155 \text{ m/s}$$

$$V_3 = V \text{ at } \frac{0.2d + 0.8d}{2} = \frac{0.1803 + 0.1532}{2} = 0.16675 \text{ m/s}$$

$$V_4 = V \text{ at } \frac{0.2d + 0.8d}{2} = \frac{0.1959 + 0.1558}{2} = 0.17585 \text{ m/s}$$

$$V_5 = V \text{ at } \frac{0.2d + 0.8d}{2} = \frac{0.1694 + 0.1584}{2} = 0.1639 \text{ m/s}$$

$$V_6 = V \text{ at } \frac{0.2d + 0.8d}{2} = \frac{0.1323 + 0.1494}{2} = 0.14085 \text{ m/s}$$

$$V_7 = V \text{ at } 0.6d = 0.2 \times 16/48 + 0.04 = 0.1067 \text{ m/s}$$

& given average depths of each section is,

$$d_1 = 1.0 \text{ m}$$

$$d_2 = 2.2 \text{ m}$$

$$d_3 = 4.0 \text{ m}$$

$$d_4 = 8.0 \text{ m}$$

$$d_5 = 4.2 \text{ m}$$

$$d_6 = 2.5 \text{ m}$$

$$d_7 = 1.2 \text{ m}$$

so, calculating the total discharge as sum of discharge from each section,

Numerical Problems

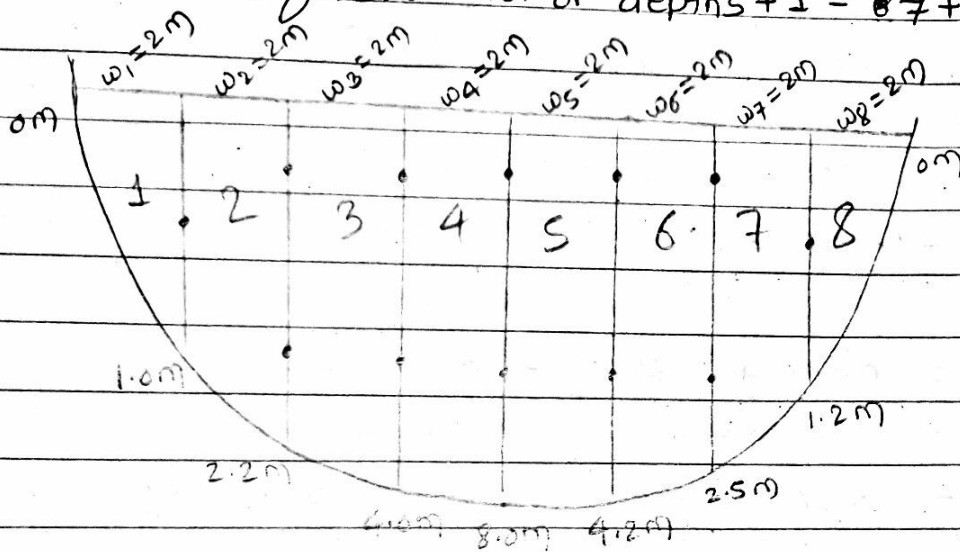
$$\begin{aligned}
 Q &= Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6 + Q_7 \\
 &= b_1 d_1 V_1 + b_2 d_2 V_2 + b_3 d_3 V_3 + b_4 d_4 V_4 + b_5 d_5 V_5 + b_6 d_6 V_6 + b_7 d_7 V_7 \\
 &= 2.25 \times 1 \times 0.09 + 2 \times 2.2 \times 0.155 + 2 \times 4 \times 0.16675 + 2 \times 8 \times 0.17585 \\
 &\quad + 2 \times 4.2 \times 0.1639 + 2 \times 2.5 \times 0.14085 + 2.25 \times 1.2 \times 0.1067
 \end{aligned}$$

$$Q = 7.4012 \text{ m}^3/\text{sec.}$$

Hence, reduced discharge is $Q = 7.4012 \text{ m}^3/\text{sec.}$

ii) By mean section method.

$$\text{no. of segment} = \text{no. of depths} + 1 = 8 + 1 = 9$$



Now, calculation of average depth of each section,

$$d_1 = \left(\frac{0 + 1}{2} \right) = 0.5 \text{ m}$$

$$d_2 = \left(\frac{1 + 2.2}{2} \right) = 1.6 \text{ m}$$

$$d_3 = \left(\frac{2.2 + 4}{2} \right) = 3.1 \text{ m}$$

Numerical Problems

$$d_4 = \left(\frac{4+8}{2} \right) = 6\text{m}$$

$$d_5 = \left(\frac{8+4.2}{2} \right) = 6.1\text{m}$$

$$d_6 = \left(\frac{4.2+2.5}{2} \right) = 3.35\text{m}$$

$$d_7 = \left(\frac{2.5+1.2}{2} \right) = 1.85\text{m}$$

$$d_8 = \left(\frac{1.2+0}{2} \right) = 0.6\text{m}$$

Then calculation of average velocities of each segment,

$$v_1 = \frac{0+0.09}{2} = 0.045\text{m/s}$$

$$v_2 = \frac{0.09+0.155}{2} = 0.1225\text{m/s}$$

$$v_3 = \frac{0.155+0.16675}{2} = 0.160875\text{m/s}$$

$$v_4 = \frac{0.16675+0.17585}{2} = 0.1713\text{m/s}$$

$$v_5 = \frac{0.17585+0.1639}{2} = 0.169875\text{m/s}$$

$$v_6 = \frac{0.1639+0.14085}{2} = 0.152375\text{m/s}$$

$$v_7 = \frac{0.14085+0.1067}{2} = 0.123775\text{m/s}$$

$$v_8 = \frac{1}{2}(0.1067+0) = 0.05335\text{m/s}$$

Numerical Problems

& average width of each section is given as,

$$b_1 = w_1 = 2 \text{ m}$$

$$b_2 = w_2 = 2 \text{ m}$$

$$b_3 = w_3 = 2 \text{ m}$$

$$b_4 = w_4 = 2 \text{ m}$$

$$b_5 = w_5 = 2 \text{ m}$$

$$b_6 = w_6 = 2 \text{ m}$$

$$b_7 = w_7 = 2 \text{ m}$$

$$b_8 = w_8 = 2 \text{ m}$$

So, calculating total discharge as sum of discharge from each section.

$$\text{i.e. } Q = Q_1 + Q_2 + Q_3 + Q_4 + Q_5 + Q_6 + Q_7 + Q_8$$

$$= b_1 d_1 V_1 + b_2 d_2 V_2 + b_3 d_3 V_3 + b_4 d_4 V_4 + b_5 d_5 V_5 + b_6 d_6 V_6 + b_7 d_7 V_7 + b_8 d_8 V_8$$

$$= 2 \times 0.5 \times 0.045 + 2 \times 1.6 \times 0.1225 + 2 \times 3.1 \times 0.160875 + 2 \times 6 \times 0.1713 + 2 \times 6.1 \times 0.169875 + 2 \times 3.35 \times 0.152375 + 2 \times 1.85 \times 0.123775 + 2 \times 0.6 \times 0.05335$$

$$\therefore Q = 7.1054 \text{ m}^3/\text{sec.}$$

Hence, required discharge is $Q = 7.1054 \text{ m}^3/\text{sec.}$

Numerical Problems.

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7 Q) The following data were collected for two verticals in a stream at a gauging station. compute the discharge in the elemental strips by,

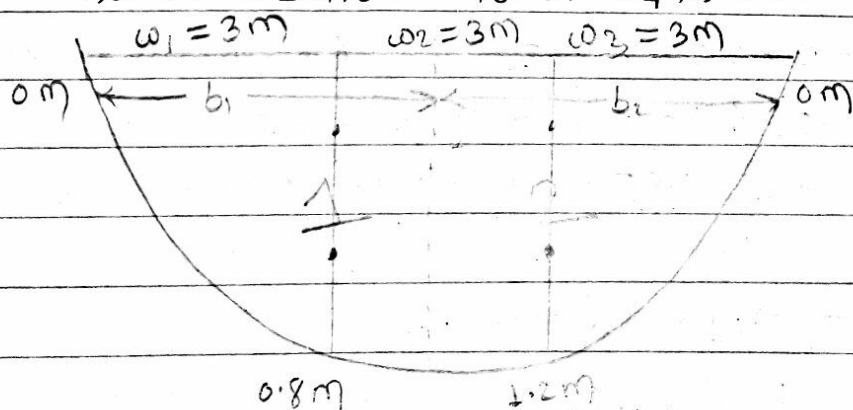
i) the mid section method ii) the mean-section method.

Distance from one end of the water surface (m)	Depth(d) (m)	Immersion of current meter below water surface (m)			
		at $0.2d$		at $0.8d$	
		rev	sec	rev	sec
3	0.8	135	150	97	151
6	1.2	150	100	150	138

Rating equation of the current meter, $V = 0.7N + 0.03$ where $N = \text{rev/sec}$ & $V = \text{velocity (m/sec)}$

Solⁿ i) By mid section method

no. of section = no. of depth = 2



Now, For the calculation of width

considering the area of both sides of the bank,

$$b_1 = \frac{(w_1 + w_2/2)^2}{2w_1} = \frac{(3 + 3/2)^2}{2 \times 3} = 3.375 \text{ m}$$

Numerical Problems.

$$b_2 = \frac{(w_3 + w_{2/2})^2}{2w_3} = \frac{(3 + 3/2)^2}{2 \times 3} = 3.375 \text{ m}$$

then, average velocities of each section is given by,

$$V_1 = V \text{ at } \frac{0.2d + 0.8d}{2} = \frac{0.66 + 0.4797}{2} = 0.56985 \text{ m/s}$$

$$V_2 = V \text{ at } \frac{0.2d + 0.8d}{2} = \frac{1.08 + 0.7909}{2} = 0.93545 \text{ m/s.}$$

& given average depths at each section is,

$$d_1 = 0.8 \text{ m}$$

$$d_2 = 1.2 \text{ m}$$

so, calculating total discharge as sum of discharges from each section.

$$\text{i.e. } Q = Q_1 + Q_2$$

$$= b_1 d_1 V_1 + b_2 d_2 V_2$$

$$= 3.375 + 0.8 \times 0.56985 + 3.375 \times 1.2 \times 0.93545$$

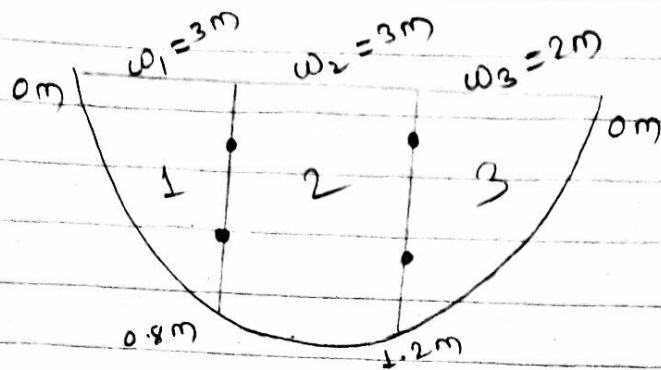
$$= 5.327 \text{ m}^3/\text{sec.}$$

Hence, required discharge is $Q = \underline{\underline{5.327 \text{ m}^3/\text{sec.}}}$

ii) By mean-section method

$$\text{no. of section} = \text{no. of depths} + 1 = 2 + 1 = 3$$

Numerical Problems



Now, calculation of average depth of each segment

$$d_1 = \left(\frac{0 + 0.8}{2} \right) = 0.4m$$

$$d_2 = \left(\frac{0.8 + 1.2}{2} \right) = 1.0m$$

$$d_3 = \left(\frac{1.2 + 0}{2} \right) = 0.6m$$

Then calculating average velocities of each segment

$$v_1 = \frac{0 + 0.56985}{2} = 0.284925 \text{ m/s}$$

$$v_2 = \frac{0.56985 + 0.93545}{2} = 0.752625 \text{ m/s}$$

$$v_3 = \frac{0.93545 + 0}{2} = 0.467725 \text{ m/s}$$

& average width of each section is given by

$$b_1 = w_1 = 3m$$

$$b_2 = w_2 = 3m$$

$$b_3 = w_3 = 3m$$

So, calculating total discharge as sum of discharges from each section.

Numerical Problems.

$$\text{i.e } Q = Q_1 + Q_2 + Q_3$$

$$= b_1 d_1 V_1 + b_2 d_2 V_2 + b_3 d_3 V_3$$

$$= 3 \times 0.4 \times 0.284925 + 3 \times 1 \times 0.752625 + 3 \times 0.6 \times 0.467725$$

$$\therefore Q = 3.44169 \text{ m}^3/\text{s}.$$

Hence, required discharge is $Q = 3.44169 \text{ m}^3/\text{s}$.

Chapter: 3 (Precipitation)

Numerical Problems.

Q.8 chait 189
16.8)

A catchment area has seven rain gauge stations.

Station	P	Q	R	S	T	U	V
Annual rainfall (cm)	130	142.1	118.2	108.5	165.2	102.1	146.9

For a 5% error in the estimation of the mean rainfall, calculate the minimum no. of additional stations required to be established in the catchment. Also find the percentage accuracy in the mean rainfall of the existing network over the basin.

Solⁿ calculating ^{average} rainfall of the whole catchment network.

$$\bar{p} = \frac{130 + 142.1 + 118.2 + 108.5 + 165.2 + 102.1 + 146.9}{7}$$

$$\therefore \bar{p} = \frac{913}{7} = 130.43 \text{ cm}$$

$$\text{Now, let } N = \text{optimum no. of raingauge station} = \left(\frac{C_v}{\%E} \right)^2$$

where, C_v = coefficient of variance

$\%E$ = percentage of error

Then,

$$\text{coefficient of variation } (C_v) = \frac{\sigma_{n-1}}{\bar{p}}$$

$$= \frac{\sqrt{\frac{\sum (P - \bar{p})^2}{n-1}}}{\bar{p}}$$

$$\text{where, } \sigma_{n-1} = \sqrt{\frac{\sum (P - \bar{p})^2}{n-1}}$$

$$= \sqrt{\frac{(-0.43)^2 + (11.67)^2 + (-12.28)^2 + (-21.93)^2 + (34.77)^2 + (-28.33)^2 + (16.47)^2}{7-1}}$$

$$= 22.54 \quad 22.55$$

$$\therefore C_v = \frac{\sigma_{n-1}}{\bar{P}} = \frac{22.545}{130.43} = 0.1729$$

As a result

$$C_v = 0.1729$$

$$\sigma_{n-1} = 22.55$$

$$\bar{P} = 130.43 \text{ cm}$$

Now,

For a 5% error, optimum no. of raingauge,

$$N = \left(\frac{C_v}{\%E} \right)^2 = \left(\frac{0.1729}{0.05} \right)^2 = 11.96 \approx 12.$$

So, additional no. of raingauge station is

$$12 - 7 = \underline{\underline{5}}$$

Also,

% of error in the average precipitation in the whole catchment area

$$\%E = \frac{C_v}{\sqrt{N}}$$

At the existing condition, $N = 7$

$$\therefore \%E = \frac{C_v}{\sqrt{N}} = \frac{0.1729}{\sqrt{7}} = 0.06535 = 6.53\%$$

Now, % of accuracy is

$$100 - 6.53 = \underline{\underline{93.47\%}}$$

Estimation of missing data. Numerical Problems.

S.2.21/18a
069 chait
J.B.

Q) The normal annual precipitation of five raingauge station P, Q, R, S & T are respectively 125, 102.76, 113 & 137 cm. During a particular storm the precipitation recorded by stations P, Q, R & S are 13.2, 9.2, 6.8 & 10.2 cm respectively. The instrument at station T was inoperative during the storm. Estimate the rainfall at station T during the storm.

Solⁿ Given,

Normal annual precipitation at station P, $N_P = 125 \text{ cm}$

Normal annual precipitation at station Q, $N_Q = 102 \text{ cm}$

Normal annual precipitation at station R, $N_R = 76 \text{ cm}$

Normal annual precipitation at station S, $N_S = 113 \text{ cm}$

Normal annual precipitation at station T, $N_T = 137 \text{ cm}$

& Annual precipitation at station P, $P_P = 13.2 \text{ cm}$

Annual precipitation at station Q, $P_Q = 9.2 \text{ cm}$

Annual precipitation at station R, $P_R = 6.8 \text{ cm}$

Annual precipitation at station S, $P_S = 10.2 \text{ cm}$

Annual precipitation at station T, $P_T = ?$

Now,

check

$$\frac{N_T - N_R}{N_T} \times 100\% = \frac{137 - 76}{137} \times 100\% \\ = 44.52\%$$

Since, normal annual rainfall of station R exceed more than 10% of the normal annual precipitation of station T. So, we use normal ratio method.

$$\text{i.e. } P_T = \frac{N_T}{N-1} \left[\frac{P_P}{N_P} + \frac{P_Q}{N_Q} + \frac{P_R}{N_R} + \frac{P_S}{N_S} \right]$$

$$\text{or, } P_T = \frac{137}{5-1} \left[\frac{13.2}{125} + \frac{9.2}{102} + \frac{6.8}{76} + \frac{10.2}{113} \right]$$

$$\therefore P_T = 12.86 \text{ cm}$$

Hence, annual precipitation at station T, $P_T = \underline{\underline{12.86 \text{ cm}}}$.

Runoff Numerical Problems

as chaitra

368) The characteristics of an isolated 1 hr storm occurred over a basin below in the table (8 marks)

% of catchment area	ϕ -index (cm/hr)	Rainfall (cm)	
		first 0.5 hr	second 0.5 hr
10	1.0	0.8	1.5
20	1.25	0.75	2.25
30	0.5	1.0	0.8
40	0.75	1.0	1.5

calculate total rainfall, total losses & runoff from the catchment.

Solⁿ For 10% catchment area, ϕ -index = 1.0 cm/hr

Time (hr)	Rainfall (cm)	Rainfall intensity ($\frac{\text{cm}}{\text{hr}}$)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	0.8	1.6	$1.6 - 1.0 = 0.6$	0.3
0.5	1.5	3	$3 - 1 = 2$	1

Total rainfall = 4.6

Total runoff = 1.3 cm

For 20% catchment area, ϕ -index = 1.25 cm/hr

Time (hr)	Rainfall (cm)	Rainfall intensity ($\frac{\text{cm}}{\text{hr}}$)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	0.75	1.5	$1.5 - 1.25 = 0.25$	0.125
0.5	2.25	4.5	$4.5 - 1.25 = 3.25$	1.625

Total rainfall = 6

Total runoff = 1.75

For 30% catchment area, ϕ -index = 0.5 cm/hr .

Time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	1.0	2.0	$2.0 - 0.5 = 1.5$	0.75
0.5	0.8	1.6	$1.6 - 0.5 = 1.1$	0.55

Total rainfall = 1.8

Total runoff = 1.3 cm

For 40% catchment area, ϕ -index = 0.75 cm/hr .

Time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	1.0	0.5 2	$2 - 0.75 = 1.25$	0.625
0.5	1.5	3	$3 - 0.75 = 2.25$	1.125

Total runoff = 1.75 cm

$$\therefore \text{Total runoff} = \frac{A_1 R_1 + A_2 R_2 + A_3 R_3 + A_4 R_4}{A_1 + A_2 + A_3 + A_4}$$

$$= \frac{10 \times 1.3 + 20 \times 1.75 + 30 \times 1.3 + 40 \times 1.75}{10 + 20 + 30 + 40}$$

$$= \frac{10 \times 1.3 + 20 \times 1.75 + 30 \times 1.3 + 40 \times 1.75}{10 + 20 + 30 + 40}$$

$$= \underline{\underline{1.57 \text{ cm}}}$$

chapter 6 Runoff & streamflow.

Numerical Problems.

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- S. 4.11) The stage & discharge data of a river are given below. Establish the stage discharge relationship to predict the discharge for a given stage. Assume the value of stage for zero discharge as 35.00m. What is the correlation coefficient of the relationship established above? Estimate the discharge corresponding to stage values of 42.50m & 48.50 respectively.

stage(m)	discharge m ³ /s	stage(m)	discharge (m ³ /s)
35.91	89	41.00	798
36.90	230	43.53	2800
37.92	360	48.02	5900
44.40	3800	49.05	6800
45.40	4560	49.55	6900
46.43	5305	49.68	6950
39.07	469		

Solⁿ Given, stage for zero discharge (a) = 35.00m

stage (G)	discharge (m ³ /s)	(G-a)	ln(G-a) = X	ln Q = Y	XY	X ²	Y ²
35.91	89	0.91	-0.0943	4.4886	-0.4233	0.0089	20.1479
36.90	230	1.9	0.6419	5.4381	3.4907	0.4120	29.5727
37.92	360	2.92	1.0716	5.8861	6.3075	1.1483	34.6462
44.40	3800	9.4	2.2407	8.2428	18.4696	5.0208	67.9430
45.40	4560	10.4	2.3418	8.4251	19.7299	5.4841	70.9819
46.43	5305	11.43	2.4362	8.5764	20.8938	5.9353	73.5547
39.07	469	4.07	1.4036	6.1506	8.6330	1.9702	37.8299

$$Q = Cx (G-a)^{\beta}$$

$$X = \beta Y + b$$

$$\text{or } \ln Q = \ln Cx + \beta \ln (G-a)$$

$$\text{or } X = \beta Y + b$$

41.00	798	6	1.7918	6.6821	11.9730	3.2104	44.6506
43.53	2800	8.53	2.1436	7.9374	17.0146	4.5950	63.0019
48.02	5900	13.02	2.5665	8.6827	22.2841	6.5869	75.3894
49.05	6800	14.05	2.6426	8.8247	23.3202	6.9835	77.8749
49.55	6900	14.55	2.6776	8.8393	23.6681	7.1695	78.1328
49.68	6950	14.68	2.6865	8.8465	23.7661	7.2172	78.2605
			$\Sigma X =$	$\Sigma Y =$	$\Sigma XY =$	$\Sigma X^2 =$	$\Sigma Y^2 =$
			24.5501	97.0204	199.1273	55.7421	751.9864

From the above table

$$\Sigma X = 24.5501 \quad \Sigma Y = 97.0204 \quad \Sigma XY = 199.1273$$

$$\Sigma X^2 = 55.7421 \quad \Sigma Y^2 = 751.9864 \quad N = 13$$

$$(\Sigma X)^2 = 602.7074 \quad (\Sigma Y)^2 = 9412.9580$$

$$\text{so, } \beta = \frac{N(\Sigma XY) - (\Sigma X)(\Sigma Y)}{N(\Sigma X^2) - (\Sigma X)^2}$$

$$= \frac{13 \times 199.1273 - 24.5501 \times 97.0204}{13 \times 55.7421 - 602.7074}$$

$$= \frac{13 \times 602.7074 - 55.7421 - 602.7074}{13 \times 55.7421 - 602.7074}$$

$$= 1.6959$$

$$\& b = \frac{\Sigma Y - \beta(\Sigma X)}{N}$$

$$N$$

$$= \frac{97.0204 - 1.6959 \times 24.5501}{13}$$

$$13$$

$$\therefore b = 4.2605 \quad \& Cx = 70.8454$$

The required gauge-discharge relationship is therefore

$$Q = 70.8454 (G-a)^{1.6959}$$

discharge at stage value 42.50m i.e $G = 42.50\text{m}$
 $Q = 70.8454 (42.50 - 35.00)^{1.6959}$
 i.e $Q = \underline{\underline{2159.3749 \text{ m}^3/\text{s}}}$

discharge at stage value 48.50m i.e $G = 48.50\text{m}$
 $Q = 70.8454 (48.50 - 35.00)^{1.6959}$
 $\therefore Q = \underline{\underline{5871.1967 \text{ m}^3/\text{s}}}$

Also, correlation coefficient

$$r = \frac{N(\sum XY) - (\sum X)(\sum Y)}{\sqrt{N(\sum X^2) - (\sum X)^2} \sqrt{N(\sum Y^2) - (\sum Y)^2}}$$

$$= \frac{13 \times 199.1273 - 24.5501 \times 97.0204}{\sqrt{13 \times 55.7421 - 602.7074} \sqrt{13 \times 751.9864 - 9412.9580}}$$

$$\therefore r = \underline{\underline{0.9831}}$$

S.4.12) Downstream of a main gauging station, an auxiliary gauge was installed and the following readings were obtained.

main gauge (m)	Auxiliary gauge (m)	discharge (m^3/s)
121.00	120.50	300
121.00	119.50	580

What discharge is indicated when the main gauge reading is 121.00 m & the auxiliary gauge reads 120.10 m.

solⁿ Given data,

$$\begin{aligned} \text{Fall (F)} &= \text{main gauge reading} - \text{auxiliary gauge reading} \\ \text{when, } F_1 &= (121 - 120.50) = 0.50 \text{ m}, & Q_1 &= 300 \text{ m}^3/\text{s} \\ F_2 &= (121 - 119.50) = 1.50 \text{ m}, & Q_2 &= 580 \text{ m}^3/\text{s} \\ F_3 &= (121 - 120.10) = 0.90 \text{ m}, & Q_3 &=? \end{aligned}$$

Now, we have,

$$\left(\frac{Q_1}{Q_2} \right) = \left(\frac{F_1}{F_2} \right)^m$$

$$\text{or, } \left(\frac{300}{580} \right) = \left(\frac{0.50}{1.50} \right)^m$$

$$\text{or, } \ln \left(\frac{300}{580} \right) = m \times \ln \left(\frac{0.50}{1.50} \right)$$

$$\therefore m = 0.6$$

$$\text{So, } \frac{Q_3}{Q_2} = \left(\frac{F_3}{F_2} \right)^m$$

$$\text{or, } Q_3 = Q_2 \left(\frac{F_3}{F_2} \right)^m$$

$$= 580 \left(\frac{0.90}{1.50} \right)^{0.6}$$

$$\therefore Q_3 = 426.877 \text{ m}^3/\text{s}.$$

Hence, discharge when main gauge reading is 121.00m & the auxiliary gauge reads 120.10m is 426.877 m³/s.

S.4.13) The following are the co-ordinates of a smooth curve drawn to best represent the stage-discharge data of a river.

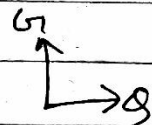
Stage (m)	20.80	21.42	21.95	23.37	23.00	23.52	23.90
discharge (m^3/s)	100	200	300	400	600	800	1000

Determine the stage corresponding to zero discharge.

Solⁿ Given, $G_1 = 20.80 \text{ m}$ $Q_1 = 100 \text{ m}^3/\text{s}$
 $G_2 = 21.42 \text{ m}$ $Q_2 = 200 \text{ m}^3/\text{s}$
 $G_3 = 21.95 \text{ m}$ $Q_3 = 300 \text{ m}^3/\text{s}$
 $G_4 = 23.37 \text{ m}$ $Q_4 = 400 \text{ m}^3/\text{s}$
 $G_5 = 23.00 \text{ m}$ $Q_5 = 600 \text{ m}^3/\text{s}$
 $G_6 = 23.52 \text{ m}$ $Q_6 = 800 \text{ m}^3/\text{s}$
 $G_7 = 23.90 \text{ m}$ $Q_7 = 1000 \text{ m}^3/\text{s}$

plot this in graph.

G vs. Q



Now, take for,

$$\frac{Q_1}{Q_2} = \frac{Q_2}{Q_3}$$

So, we assume, $Q_1 = 200 \text{ m}^3/\text{s}$
 $Q_2 = 400 \text{ m}^3/\text{s}$
 $Q_3 = 800 \text{ m}^3/\text{s}$

$$\text{So, } \frac{Q_1}{Q_2} = \frac{200}{400} = \frac{1}{2} \quad \text{So, } \frac{Q_2}{Q_3} = \frac{400}{800} = \frac{1}{2}$$

So, value of stage corresponding to zero discharge is calculated as,

$$G_1 = 21.42 \text{ m}, G_2 = 23.37 \text{ m}, G_3 = 23.52 \text{ m}$$

So, value of stage corresponding to zero discharge is calculated as,

$$a = \frac{G_1 G_3 - G_2^2}{G_1 + G_3 - 2G_2}$$

$$\text{or, } a = \frac{21.42 \times 23.52 - (23.37)^2}{21.42 + 23.52 - 23.37 \times 2}$$

$$\therefore a = 23.53 \text{ m}$$

Hence, At zero discharge, stage value is, $a = 23.53 \text{ m}$

5.4.15) During a flood the water surface at a section in a river was found to increased at a rate of 11.2 cm/hr . The slope of the river is $\frac{1}{3600}$ & the normal discharge for the river stage read from a steady-flow rating curve was $160 \text{ m}^3/\text{s}$. If the velocity of the flood wave can be assumed to be 2.0 m/s , determine the actual discharge.

solⁿ Given,

$$\begin{aligned} \text{rate of change of stage } (dh/dt) &= 11.2 \text{ cm/hr} \\ &= \frac{11.2 \times 10^{-2} \text{ m}}{3600 \text{ sec}} \\ &= 3.11 \times 10^{-5} \text{ m/s.} \end{aligned}$$

$$\text{bed slope } (S_0) = \frac{1}{3600}$$

$$\text{normal discharge } (Q_n) = 160 \text{ m}^3/\text{s.}$$

$$\text{velocity of flood wave } (V_w) = 2.0 \text{ m/s}$$

$$\text{actual discharge } (Q_m) = ?$$

Now, using the relation,

$$\frac{Q_m}{Q_n} = \sqrt{1 + \frac{1}{V_w S_0} \frac{dh}{dt}}$$

$$\text{or, } Q_m = Q_n \sqrt{1 + \frac{1}{V_w S_0} \frac{dh}{dt}}$$

$$\text{or, } Q_m = 160 \sqrt{1 + \frac{0.112}{3600 \times \frac{1}{3600}}}$$

$$\text{or, } Q_m = 160 \sqrt{1 + \frac{0.112}{2}}$$

$$\text{or, } Q_m = 164.42 \text{ m}^3/\text{s}.$$

Hence, actual discharge of the flow is $Q = 164.42 \text{ m}^3/\text{s}$.

ex 8.4.10

S. 8.4.10) A stream has a trapezoidal cross section with base width of 12m and side slope 2 horizontal : 1 vertical in a reach of 8 km. During a flood the high water levels record at the ends of the reach are as follows.

section	Elevation of bed (m)	water surface elevation (m)	Remarks
upstream	100.20	102.70	Manning's $n = 0.030$
downstream	98.60	101.30	

Estimate the discharge in the stream.

solⁿ using suffixes 1 & 2 to denote the upstream & downstream sections respectively.

Given, $b_1 = b_2 = 12\text{m}$, $z = 2$, $L = 8000\text{m}$

the cross-sectional properties are calculated as follows.

Section 1	Section 2
$b_1 = 12\text{m}$	$b_2 = 12\text{m}$
$y_1 = 102.70 - 100.20 = 2.5\text{m}$	$y_2 = 101.30 - 98.60 = 2.7\text{m}$
$\therefore A_1 = b_1 y_1 + z y_1^2$	$\therefore A_2 = b_2 y_2 + z y_2^2$
$= 12 \times 2.5 + 2 \times (2.5)^2$	$= 12 \times 2.7 + 2 \times (2.7)^2$
$= 42.5\text{m}^2$	$= 46.98\text{m}^2$
$\& P_1 = b_1 + 2y_1 \sqrt{1+z^2}$	$\& P_2 = b_2 + 2y_2 \sqrt{1+z^2}$
$= 12 + 2 \times 2.5 \sqrt{1+4}$	$= 12 + 2 \times 2.7 \sqrt{1+4}$
$= 23.18\text{m}$	$= 24.07\text{m}$
$\therefore R_1 = \frac{A_1}{P_1} = \frac{42.5}{23.18} = 1.8335\text{m}$	$\therefore R_2 = \frac{A_2}{P_2} = \frac{46.98}{24.07} = 1.9518\text{m}$
$\therefore K_1 = \frac{1}{n} A_1 R_1^{2/3}$	$\therefore K_2 = \frac{1}{n} A_2 R_2^{2/3}$
$= \frac{1}{0.03} \times (42.5) \times (1.8335)^{2/3}$	$= \frac{1}{0.03} \times (46.98) \times (1.9518)^{2/3}$
$= 2122.21$	$= 2445.77$

Average K for the reach, $K = \sqrt{K_1 K_2}$

$$= \sqrt{2122.21 \times 2445.77}$$

$$= 2278.25$$

To start with $h_f = (h_1 - h_2) = 102.70 - 101.30 = 1.4\text{m}$
 Eddy loss, $h_e = 0$, as $K_e = 0$ for uniform width

The calculations are conducted as,

$$\bar{S}_F = h_F/L = \frac{h_F}{8000}$$

$$Q = K\sqrt{\bar{S}_F} = 2278.25\sqrt{\bar{S}_F}$$

$$\frac{v_1^2}{2g} = \left(\frac{Q}{42.5}\right)^2 \times \frac{1}{19.6}$$

$$\frac{v_2^2}{2g} = \left(\frac{Q}{46.98}\right)^2 \times \frac{1}{19.6}$$

$$h_F = (h_1 - h_2) + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right) - h_e$$

$$\therefore h_F = 1.4 + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right)$$

Trial	h_F (trial) (m)	S_F (units of 10^{-4})	Q (m^3/s)	$v_1^2/2g$ (m)	$v_2^2/2g$ (m)	h_F (new)
1	1.4	1.75	30.1384	0.0257	0.0210	1.4047
2	1.4047	1.7559	30.1892	0.0257	0.0211	1.4046
3	1.4046	1.75575	30.1879	0.0257	0.0211	1.4046

Since value of h_F is same, so we stop trial.

Hence, the discharge in the stream is $Q = 30.1879 m^3/s$.

S. 4.9)

Solⁿ Given, length (L) = 10000 m
the cross sectional properties are calculated as,

Section A	Section B
$h_1 = 104.771 \text{ m}$	$h_2 = 104.500 \text{ m}$
$A_1 = 73.293 \text{ m}^2$	$A_2 = 93.375 \text{ m}^2$
$R_1 = 2.733 \text{ m}$	$R_2 = 3.089 \text{ m}$
$K_1 = \frac{1}{n} A_1 R_1^{2/3}$	$K_2 = \frac{1}{n} A_2 R_2^{2/3}$
$= \frac{1}{0.02} \times (73.293) \times (2.733)^{2/3}$	$= \frac{1}{0.02} \times (93.375) \times (3.089)^{2/3}$
$= 7163.50$	$= 9902.52$

Average K of the reach, $K = \sqrt{K_1 K_2}$
 $= \sqrt{7163.50 \times 9902.52}$
 $= 8422.39$

To start, $h_F = (h_1 - h_2) = 104.771 - 104.500 = 0.271 \text{ m}$

Eddy loss, $h_e = K_e \left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right] = 0.3 \left[\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right]$ (for expansion channel)

The calculations are conducted as,

$$S_F = h_F / L = \frac{h_F}{10000}$$

$$Q = K \sqrt{S_F} = 8422.39 \sqrt{S_F}$$

$$\frac{V_1^2}{2g} = \left(\frac{Q}{73.293} \right)^2 \times \frac{1}{19.62}$$

$$\frac{V_2^2}{2g} = \left(\frac{Q}{93.375} \right)^2 \times \frac{1}{19.62}$$

$$\therefore h_F = (h_1 - h_2) + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right) - K_e \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g} \right)$$

$$\text{i.e } h_f = 0.271 + \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} \right) - 0.3 \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} \right)$$

Trial	$h_f(\text{trial})$ (m)	SF (units of 10^{-5})	Q (m^3/s)	$\frac{v_1^2}{2g}$ (m)	$\frac{v_2^2}{2g}$ (m)	$h_f(\text{new})$
1	0.271	2.71	43.845	0.0182	0.0112	0.2759
2	0.2759	2.759	44.2396	0.0186	0.0114	0.2760 0.2747
3	0.2760	2.760	44.2476	0.0186	0.0114	0.2760

Since, value of h_f is same so we stop trial.

Hence, the discharge in the stream is $Q = \underline{\underline{44.2476 \text{ m}^3/\text{s}}}$.

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Numerical problems.

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3b) An isolated 3-hr storm occurred over a basin in the following fashion.

% of catchment area (km ²)	ϕ -index (cm/hr)	Rainfall (cm)		
		1 st hr	2 nd hr.	3 rd hr.
20	1.00	0.8	2.3	1.5
30	0.75	0.7	2.1	1.0
50	0.50	1.0	2.5	0.8

solⁿ For 20% catchment area, $\phi = 1.00$ cm/hr

time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	Excess rainfall (cm)
1	0.8	$0.8/1 = 0.8$	$0.8 - 1 = -0.2$	0
1	2.3	$2.3/1 = 2.3$	$2.3 - 1 = 1.3$	1.3
1	1.5	$1.5/1 = 1.5$	$1.5 - 1 = 0.5$	0.5

Total runoff (R_1) = 1.8 cm

For 30% catchment area, $\phi = 0.75$ cm/hr.

time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
1	0.7	$0.7/1 = 0.7$	$0.7 - 0.75 = -0.05$	0
1	2.1	$2.1/1 = 2.1$	$2.1 - 0.75 = 1.35$	1.35
1	1.0	$1.0/1 = 1.0$	$1.0 - 0.75 = 0.25$	0.25

Total runoff (R_2) = 1.60 cm

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For 50% catchment area, $\phi = 0.50 \text{ cm/hr}$.

time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
1	1.0	$1.0/1 = 1.0$	$1.0 - 0.50 = 0.50$	0.50
1	2.5	$2.5/1 = 2.5$	$2.5 - 0.50 = 2.0$	2.0
1	0.8	$0.8/1 = 0.8$	$0.8 - 0.5 = 0.3$	0.3

Total runoff (R_3) = 2.80 cm

Now,

$$\begin{aligned} \text{Total runoff from catchment (R)} &= \frac{A_1 R_1 + A_2 R_2 + A_3 R_3}{A_1 + A_2 + A_3} \\ &= \frac{20 \times 1.8 + 30 \times 1.6 + 50 \times 2.8}{20 + 30 + 50} \end{aligned}$$

$$\text{i.e. } R = \underline{\underline{2.24 \text{ cm}}}$$

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Q 3.6. g) The characteristics of an isolated 1 hr storm occurred over a basin is given below in the table.

% of catchment area	ϕ Index (cm/hr)	Rainfall (cm)	
		first 0.5 hr	second 0.5 hr
10	1.0	0.8	1.5
20	1.25	0.75	2.25
30	0.5	1.0	0.8
40	0.75	1.0	1.5

Soln For 10% catchment area, $\phi = 1.0 \text{ cm/hr}$

ch. 84

1	2	3 = 2/1	4 = 3 - ϕ	5 = 4 x 1
time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	0.8	1.6	1.6 - 1 = 0.6	0.3
0.5	1.5	3.0	3.0 - 1 = 2.0	1.0

for 20% catchment area, $\phi = 1.25 \text{ cm/hr}$. Total runoff (R_1) = 1.3 cm

time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	0.75	1.5	1.5 - 1.25 = 0.25	0.125
0.5	2.25	4.5	4.5 - 1.25 = 3.25	1.625

Total runoff (R_2) = 1.75 cm

for 30% catchment area, $\phi = 0.50 \text{ cm/hr}$.

time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	1.0	2	2 - 0.5 = 1.50	0.75
0.5	0.8	1.6	1.6 - 0.5 = 1.10	0.55

Total runoff (R_3) = 1.30 cm

for 40% catchment area, $\phi = 0.75 \text{ cm/hr}$

time (hr)	Rainfall (cm)	Rainfall intensity (cm/hr)	effective rainfall intensity (cm/hr)	excess rainfall (cm)
0.5	1.0	2	2 - 0.75 = 1.25	0.625
0.5	1.5	3	3 - 0.75 = 2.25	1.125

Total runoff (R_4) = 1.75 cm

Now, Total runoff from catchment (R) = $\frac{A_1 R_1 + A_2 R_2 + A_3 R_3 + A_4 R_4}{A_1 + A_2 + A_3 + A_4}$

$$= \frac{10 \times 1.3 + 20 \times 1.75 + 30 \times 1.30 + 40 \times 1.75}{10 + 20 + 30 + 40} = 1.57 \text{ cm}$$

ch. 8

07/11/2019

Q) For a drainage basin of 600 km^2 , isohyets drawn for a storm gave the following data.

Isohyets (interval) (cm)	15-12	12-9	9-6	6-3	3-1
Inter-isohyetal area (km^2)	92	128	120	175	85

solⁿ The calculation are conducted as,

Isohytes (cm)	Average value (P_n) (cm)	Area (A_n) (km^2)	$A_n \times P_n$ ($\text{cm} \cdot \text{km}^2$)
15-12	13.5	92	1242
12-9	10.5	128	1344
9-6	7.5	120	900
6-3	4.5	175	787.5
3-1	2.0	85	170
Total.		$\Sigma A_n = 600$	$\Sigma A_n P_n = 4443.5$

Now,

$$\text{Average depth of precipitation } (\bar{p}) = \frac{\Sigma A_n P_n}{\Sigma A_n}$$

$$= \frac{4443.5}{600}$$

$$\therefore \bar{p} = \underline{\underline{7.41 \text{ cm}}}$$

chapter:- 7 Hydrograph Analysis.

7.1) Hydrograph : concept & components.

Hydrograph is a graphical plot between discharge (Q) of a river at a given location over time.

* Hydrograph components.

MA = base flow

AB = rising limb

BC = crest segment

CD = falling limb

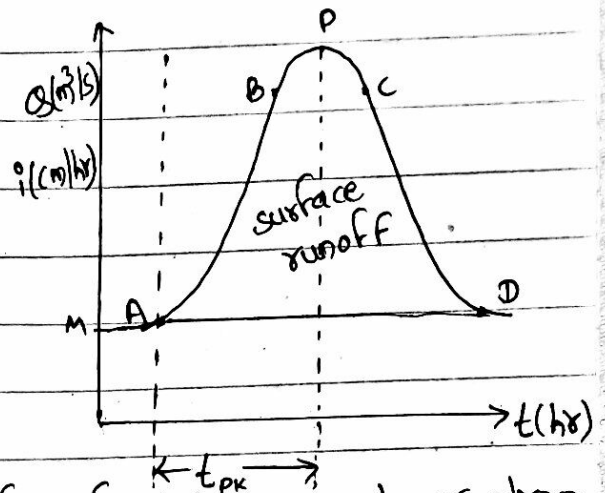


fig:- Flood hydrograph or storm hydrograph or single storm due to isolated storm.

Points

A \rightarrow starting point of rising curve

B & C \rightarrow points of inflections

P \rightarrow peak discharge points

D \rightarrow end point of falling limb

T_b \rightarrow time base of hydrograph

T_{pk} \rightarrow time to peak

* Separation of base flow

07J chaitra 8) Write the base flow separation method.

\Rightarrow The base flow separation method are given as, of three types which are given as,

i) straight line method

Hydrology chapter. 9. Hydrology of floods

072 chaitra:

Solⁿ Given,

<u>Return period (years)</u>	<u>Peak Flood (m³/s)</u>
100	435
50	395
1000	?

Now, using Gumbel equation,

$$Q_T = \bar{Q} + \sigma_{n-1} \cdot K_T$$

$$\text{or, } Q_{100} = \bar{Q} + \sigma_{n-1} \cdot K_{100}$$

$$\text{i.e. } 435 = \bar{Q} + \sigma_{n-1} \cdot K_{100} \quad \text{--- ①}$$

$$\& \quad Q_{50} = \bar{Q} + \sigma_{n-1} \cdot K_{50}$$

$$\text{i.e. } 395 = \bar{Q} + \sigma_{n-1} \cdot K_{50} \quad \text{--- ②}$$

Solving equation ① & ② we get

$$435 = \bar{Q} + \sigma_{n-1} \cdot K_{100}$$

$$395 = \bar{Q} + \sigma_{n-1} \cdot K_{50}$$

$$40 = \sigma_{n-1} (K_{100} - K_{50})$$

$$= \sigma_{n-1} \left(\frac{Y_{100} - Y_{50}}{S_n} \right)$$

Since,

$$Y_{100} = -\ln \ln \left(\frac{100}{100-1} \right) = 4.6001$$

$$\& \quad Y_{50} = -\ln \ln \left(\frac{50}{50-1} \right) = 3.9019$$

so, we get

$$\frac{\sigma_{n-1}}{s_n} = \frac{40}{(Y_{100} - Y_{50})}$$
$$= \frac{40}{4.6001 - 3.9019}$$

$$\text{or, } \frac{\sigma_{n-1}}{s_n} = 57.2902 \quad \text{--- (ii)}$$

$$\text{Now, } Q_{1000} = \bar{Q} + \sigma_{n-1} \cdot K_{1000} \quad \text{--- (i)}$$

from equation (i) & (iv) we get

$$435 = \bar{Q} + \sigma_{n-1} \cdot K_{100}$$

$$Q_{1000} = \bar{Q} + \sigma_{n-1} \cdot K_{1000}$$

$$435 - Q_{1000} = \sigma_{n-1} (K_{100} - K_{1000})$$

$$\text{or, } Q_{1000} = 435 - \sigma_{n-1} (K_{100} - K_{1000})$$
$$= 435 - \sigma_{n-1} \left(\frac{Y_{100} - Y_{1000}}{s_n} \right)$$

$$= 435 - \frac{\sigma_{n-1}}{s_n} (Y_{100} - Y_{1000})$$

$$\text{since, } Y_{1000} = -\ln \ln \left(\frac{1000}{1000-1} \right) = 6.9073$$

$$\text{so, } Q_{1000} = 435 - 57.2902 (4.6001 - 6.9073) \quad \text{from (ii)}$$

$$\text{or, } Q_{1000} = 435 + 132.1799$$

$$\text{i.e. } Q_{1000} = 567.18 \text{ m}^3/\text{s.}$$

Hence discharge in given river having return period of 1000 years is 567.18 m³/sec.

o/o chaitra.

Solⁿ Given,

<u>Return period (years)</u>	<u>Peak Flood (m³/s)</u>
100	435
50	395
500	?

Now, using Gumbel equation,

$$Q_T = \bar{Q} + \sigma_{n-1} \cdot K_T$$

$$\text{or, } Q_{100} = \bar{Q} + \sigma_{n-1} \cdot K_{100}$$

$$\text{i.e. } 435 = \bar{Q} + \sigma_{n-1} \cdot K_{100} \quad \text{--- ①}$$

$$\& \quad Q_{50} = \bar{Q} + \sigma_{n-1} \cdot K_{50}$$

$$\text{i.e. } 395 = \bar{Q} + \sigma_{n-1} \cdot K_{50} \quad \text{--- ②}$$

From equation ① & ②

$$435 = \bar{Q} + \sigma_{n-1} \cdot K_{100}$$

$$395 = \bar{Q} + \sigma_{n-1} \cdot K_{50}$$

$$40 = \sigma_{n-1} (K_{100} - K_{50})$$

$$\text{or, } 40 = \sigma_{n-1} \left(\frac{\gamma_{100} - \gamma_{50}}{s_n} \right)$$

$$\text{or, } \frac{\sigma_{n-1}}{s_n} = \frac{40}{(\gamma_{100} - \gamma_{50})}$$

Since

$$\gamma_{100} = -\ln \ln \left(\frac{100}{100-1} \right) = 4.6001$$

$$\& \quad \gamma_{50} = -\ln \ln \left(\frac{50}{50-1} \right) = 3.9019$$

so we get

$$\frac{\sigma_{n-1}}{s_n} = \frac{40}{(4.6001 - 3.9019)}$$

$$\text{i.e. } \frac{\sigma_{n-1}}{s_n} = 57.2902 \text{ --- (iii)}$$

Now,

$$Q_{500} = \bar{Q} + \sigma_{n-1} \cdot K_{500} \text{ --- (iv)}$$

Using equation (ii) & (iv) we get

$$395 = \bar{Q} + \sigma_{n-1} \cdot K_{50}$$

$$Q_{500} = \bar{Q} + \sigma_{n-1} \cdot K_{500}$$

$$395 - Q_{500} = \sigma_{n-1} (K_{50} - K_{500})$$

$$\text{or, } Q_{500} = 395 - \sigma_{n-1} (K_{50} - K_{500})$$

$$\text{or, } Q_{500} = 395 - \sigma_{n-1} \left(\frac{Y_{50} - Y_{500}}{s_n} \right)$$

Since,

$$Y_{500} = -\ln \ln \left(\frac{500}{500-1} \right) = 6.2136$$

$$\text{so, } Q_{500} = 395 - \frac{\sigma_{n-1}}{s_n} (3.9019 - 6.2136)$$

$$= 395 - 57.2902 (3.9019 - 6.2136) \text{ from iii}$$

$$\text{or, } Q_{500} = 527.44 \text{ m}^3/\text{s}.$$

Hence, discharge of a given river having return period of 500 years is 527.44 m³/s.

071 Chaitra

Solⁿ Given,

$$N = 92 \text{ years}$$

$$\text{mean, } \bar{Q} = 6437 \text{ m}^3/\text{sec}$$

$$\text{standard deviation, } \sigma_{n-1} = 2951 \text{ m}^3/\text{sec}.$$

Flood discharge with a return period 500 years,
 $Q_{500} = ?$

Now, using table for $N = 92$ years,

$$\text{Reduced mean, } \bar{Y}_n = 0.5589$$

$$\text{Reduced standard deviation, } S_n = 1.2020$$

i) Using Gumbel's equation

$$Q_T = \bar{Q} + \sigma_{n-1} \cdot K_T$$

$$\text{or, } Q_{500} = \bar{Q} + \sigma_{n-1} \cdot K_{500}$$

$$\text{Since, } K_{500} = \left(\frac{Y_{500} - \bar{Y}_n}{S_n} \right)$$

$$\& Y_{500} = -\ln \ln \left(\frac{500}{500-1} \right) = 6.2136$$

so,

$$K_{500} = \left(\frac{6.2136 - 0.5589}{1.2020} \right)$$

$$= 4.7044$$

so, we get

$$\begin{aligned} Q_{500} &= 6437 + 2951 \times 4.7044 \\ &= 20319.68 \text{ m}^3/\text{sec}. \end{aligned}$$

ii) For confidence limit, $f(c) = 1.96$ (given)

Now, we have,

$$Q_{1/2} = Q_T \pm f(c) S_e$$

where,

$$S_e = \frac{b \sigma_{n-1}}{\sqrt{N}}$$

$$= \frac{\sqrt{1 + 1.3 K_{500} + 1.1 K_{500}^2} \cdot \sigma_{n-1}}{\sqrt{92}}$$

$$= 2951 \sqrt{\frac{1 + 1.3 \times (4.7044) + 1.1 \times (4.7044)^2}{92}}$$

$$= 1725.6642$$

$$\text{So, } Q_{1/2} = 20319.68 \pm 1.96 \times 1725.6642$$

$$= 20319.68 \pm 3382.30$$

$$\text{i.e. } Q_1 = 20319.68 + 3382.30 = 23701.98 \text{ m}^3/\text{sec.}$$

$$Q_2 = 20319.68 - 3382.30 = 16937.38 \text{ m}^3/\text{sec.}$$

iii) For discharge adopted, safety factor = 1.3

Now,

$$\begin{aligned} \text{adopted discharge} &= 1.3 \times \text{recommended discharge} \\ &= 1.3 \times 20319.68 \\ &= 26415.58 \text{ m}^3/\text{s} \end{aligned}$$

Also, for safety margin,

$$\begin{aligned} \text{safety margin} &= \text{adopted discharge} - \text{recommended discharge} \\ &= 26415.58 - 20319.68 \end{aligned}$$

$$= 6095.90 \text{ m}^3/\text{sec.} //$$

069 chaitra

Sol: Given, discharge, $Q_T = 350 \text{ m}^3/\text{s}$.

$$\text{mean } (\bar{Q}) = 121 \text{ m}^3/\text{sec}$$

$$\text{Standard deviation } (\sigma_{n-1}) = 60 \text{ m}^3/\text{sec}.$$

For, $N = 20$ years,

$$\text{Reduced mean } (\bar{y}_n) = 0.5236$$

$$\text{Reduced standard deviation } (s_n) = 1.0628$$

expected life $(n) = 30$ years,

Now, calculating return period of given flood,
using Gumbel's equation,

$$Q_T = \bar{Q} + \sigma_{n-1} \cdot K_T$$

$$\text{or, } 350 = 121 + 60 \times K_T$$

$$\text{or, } 60 K_T = 350 - 121$$

$$\text{or, } K_T = \frac{229}{60}$$

$$\text{or, } K_T = 3.8167$$

$$\text{or, } \frac{Y_T - \bar{y}_n}{s_n} = 3.8167$$

$$\text{or, } \frac{Y_T - \cancel{0.5236}}{1.0628 \cancel{0.5236}} = 3.8167$$

$$\text{or, } Y_T = 3.8167 \times 1.0628 + \cancel{0.5236} \times 0.5236$$

$$\text{or, } Y_T = 4.58$$

$$\text{or, } -\ln \ln \left(\frac{T}{T-1} \right) = 4.58$$

$$\text{or, } \ln \ln \left(\frac{T}{T-1} \right) = -4.58$$

$$\text{or, } \ln\left(\frac{T}{T-1}\right) = \cancel{e^{-4.58}} e^{-4.58}$$

$$\text{or, } \ln\left(\frac{T}{T-1}\right) = 0.0102$$

$$\text{or, } \frac{T}{T-1} = e^{0.0102}$$

$$\text{or, } \frac{T}{T-1} = 1.0102$$

$$\text{or, } T = 1.0102T - 1.0102$$

$$\text{or, } 1.0102 = (1.0102 - 1)T$$

$$\text{or, } T = 99 \text{ years}$$

$$\begin{aligned} \text{Then, Risk, } \bar{R} &= 1 - \left(1 - \frac{1}{T}\right)^n \\ &= 1 - \left(1 - \frac{1}{99}\right)^{30} \\ &= 0.2626 \end{aligned}$$

$$\text{i.e Risk, } \bar{R} = 26.26\%$$

Hence Risk value of the given structure is, $\bar{R} = 26.26\%$.

068 chapter

Sol? Given,

$$\text{mean } (\bar{Q}) = 8520 \text{ m}^3/\text{sec.}$$

$$\text{standard deviation } (\sigma_{n-1}) = 3900 \text{ m}^3/\text{sec.}$$

$$\text{expected life } (n) = 40 \text{ years}$$

$$\text{Reliability } (R_e) = 85\% = 0.85$$

for $N = 21$ years, given,

$$\text{Reduced mean } (\bar{y}_n) = 0.5252$$

$$\text{Reduced standard deviation } (s_n) = 1.0696$$

$$\text{Flood discharge } (Q_T) = ?$$

Now,

using Gumbel equation,

$$Q_T = \bar{Q} + \sigma_{n-1} \cdot K_T$$

$$\text{since } K_T = \frac{y_T - \bar{y}_n}{s_n}$$

for T ,

$$R_e = (1 - 1/T)^n$$

$$\text{or, } 0.85 = (1 - 1/T)^{40}$$

$$\text{or, } (1 - 1/T) = 0.9959$$

$$\therefore T = 246.6255 \text{ years}$$

then,

$$K_T = y_T = -\ln \ln \left(\frac{T}{T-1} \right)$$

$$= -\ln \ln \left(\frac{246.6255}{245.6255} \right) = 5.5058$$

$$\text{then, } K_T = \frac{Y_T - \bar{Y}_n}{S_n} = \frac{5.5058 - 0.5252}{1.0696} = 4.6565$$

so,

$$\begin{aligned} \text{a) } \bar{Q}_T &= \bar{Q} + \sigma_{n-1} K_T \\ &= 8520 + 3900 + 4.6565 \\ &= 26680.35 \text{ m}^3/\text{sec.} \end{aligned}$$

b) If safety factor = 1.3, adopted discharge = ?
Now,

$$\begin{aligned} \text{adopted discharge} &= 1.3 \times \text{recommended discharge} \\ &= 1.3 \times 26680.35 \\ &= 34684.455 \text{ m}^3/\text{sec.} \end{aligned}$$

And

$$\begin{aligned} \text{safety margin} &= \text{adopted discharge} - \text{recommended discharge} \\ &= 34684.455 - 26680.35 \\ &= 8004.105 \text{ m}^3/\text{sec.} \end{aligned}$$